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HIGHWAY RESEARCH RECORD

Number 371 | Highway Design Practices

7 Reports

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Subject Area

53 Traffic Control and Operations

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CONTENTS

FOREWORD

Each paper in this RECORD approaches the subject of design practices from a different perspective, but all of the papers are oriented toward improvement in capacity and safety.

The paper by Stimpson and Glennon makes note of the increasing concern by highway design engineers regarding the suitability of present climbing lane design practices for the high-speed operation on major highways. After a critical review of current design procedures, the authors illustrate a number of inadequacies and present specific recommendations for design improvement.

Research reported by Haefner and Morlok concentrated on the relationship between geometric design and accident levels. A decision-making framework is suggested to assist the highway engineer in making decisions relative to safety improvements.

Pendakur and Roer examined the adequacy of parking at 11 public general hospitals in metropolitan Vancouver. The main focus of the study was to describe and understand the traffic and parking needs with a view to developing overall planning standards for access and parking.

McHenry, Deleys, and Eicher describe a mathematical model that represents the dynamic motions of a motor vehicle in contact with a variety of roadway and roadside geometric design features including fixed objects.

Smith, Yotter, and Murphy discuss a technique wherein perspective drawings of a proposed or existing roadway are produced by computer. They offer rules to guide highway design engineers in coordinating horizontal and vertical elements of highway alignment.

The summary of the conference session on TOPICS reviews the background and purpose of the Traffic Operations Program to Increase Capacity and Safety, which is aimed at relieving existing hazards and traffic bottlenecks in urban areas. This summary also contains a discussion of techniques for evaluating the effectiveness of geometric improvements made under the TOPICS.

Ring and Carstens present techniques that can be used in making decisions for installing left-turn lanes at rural intersections. The procedures are based on benefit-cost relationships.

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CRITICAL REVIEW OF CLIMBING-LANE DESIGN PRACTICES

William A. Stimpson and John C. Glennon, Texas Transportation Institute, Texas A&M University

There is an increasing concern by highway design engineers regarding the suitability of design practices for climbing lanes on high-speed major highways. This report endorses the conclusions in an earlier report (2) and attempts to expand on them to achieve a more comprehensive design procedure. A critical review of current design procedures is, of course, a prerequisite to more specific design recommendations. The authors assume that the reader has a basic familiarity with the traditional approaches that use AASHO policy as a key tool. Truck grade preference determination for a proposed profile, speed reduction criteria, and capacity as a design warrant are the essential aspects of the procedural review. Methods currently in use were deemed inadequate. Specific design recommendations are offered.

• HIGHWAYS should be designed to encourage uniform speed throughout. Using a selected design speed and correlating various geometric elements are means toward this end. Design values have been determined and agreed upon for many highway elements, but few conclusions have been reached regarding the relation between roadway grades and design speed (1).

It cannot be denied that several studies have been valuable in quantifying the vast operating differences among various classes of vehicles on grades. All highway engineers realize that freedom of operation for traffic in rolling and mountainous terrain, particularly on 2-lane roads, is greatly affected by the relatively poor performance of heavy trucks or grades. It is not difficult to find undesirable situations created in spite of our knowledge of truck performance on grades. There are countless examples of inefficient and unsafe operation. They can be found on new facilities as well as on many old ones. The difficulties are most severe in the vicinity of summits where sight distances are often limited and truck speeds are at their minimum.

DESIGN CRITERIA

In the design of highway grades, consideration is given to the "critical length of grade." This is the length of a particular percentage of grade that will cause a designated design vehicle to operate at some predetermined minimum speed. A lower speed is considered unacceptable for safety and operational efficiency but only if the design hourly volume exceeds the design capacity of the grade by 20 percent on 2-lane highways or 30 percent on multilane highways. Two alternatives are usually considered when a designed grade is longer than critical: (a) Adjust the grade line until it is no longer critical, or (b) add an auxiliary climbing lane in which slowly moving vehicles can operate adjacent to the main travel lane.

The AASHO Policy (1) suggests that climbing lanes are necessary when the length of a specific grade causes truck speeds to reduce 15 mph or more, provided that the volume

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of traffic and percentage of heavy trucks justify the added cost. Therefore, both truck performance on grades and highway capacity are employed as climbing-lane design warrants.

Grade Performance Determination

The AASHO Policy recommends the use of Huff and Scrivner's speed-distance curves (3), which were developed from road tests in 1953. These curves, shown in Figure 1, appear to depict an appropriate level of performance for design purposes. They were developed for a design vehicle with an approximate weight-horsepower ratio of 400:1, which represents a suitable upper boundary for trucks currently on the highways.

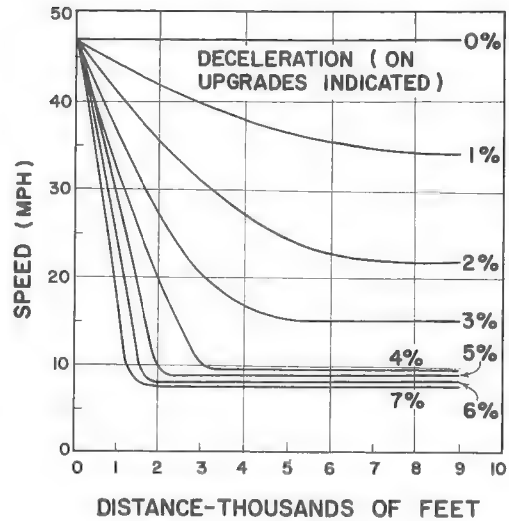

Figure 1. Speed-distance curves for a heavy truck operating on various grades.

Speed Reduction Criterion

One of those intangibles that can and should be directly related to design practice is the accident potential of various grades. It is obviously impossible to examine accident records on proposed new construction. There is, however, a strong need to establish a speed-reduction criterion that appropriately reflects the seriousness of large speed differentials and consequential accident involvement.

Glennon (2) has proposed a method to fill this need, and thereby, to put the design problem on a broader analytical base. In an analysis of expected truck accident involvement rates for various speed reductions of the design truck on highway grades, he developed the relationship shown in Figure 2. This relationship indicates that a design based on a 15-mph speed-reduction criterion is likely to yield more than twice as many truck accidents than one based on a 10-mph criterion. In addition, more serious and costly accidents can be expected when the speed differential of the involved vehicles is greater. The adoption of a 10-mph speed-reduction criterion was, therefore, recommended for climbing-lane design. There is greater justification for this improved criterion than any heretofore proposed.

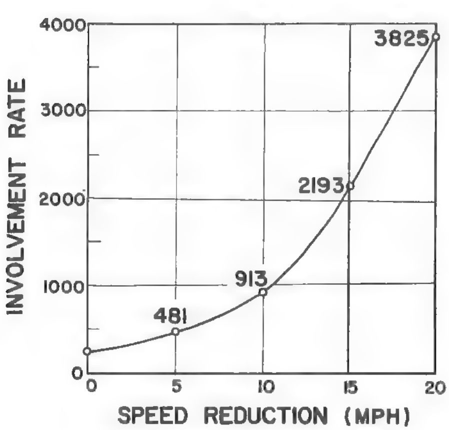

Figure 2. Truck accident involvement rate related to speed reduction of design vehicle.

Capacity Criteria

After the analysis has been made of the critical length of grade, the justification for a climbing lane is typically considered from the standpoint of highway capacity. The AASHO Policy states that, regardless of the grade performance of the design truck on a particular highway grade, a climbing lane is not warranted unless the design hourly volume (DHV) exceeds 120 percent of the design capacity of a 2-lane highway grade or 130 percent of the design capacity of 1 direction of a multilane highway grade.

Because desired automobile speeds have increased in recent years at a faster rate than truck speeds, the burden of grade congestion on highway users is worsening. It does not appear logical, under present traffic conditions,

to deliberately allow a hazardous bottleneck to be "built" into a highway. It, therefore, seems appropriate to eliminate the capacity overrun criteria. That is, climbing lanes should be warranted from a capacity standpoint if the DHV equals or exceeds the design capacity of the grade.

ANALYSIS PROCEDURES

Grade Performance Analysis

In using the curves shown in Figure 1, the engineer is always working with differential distances. Unless the curves are plotted to a very large scale, these distances are difficult to determine accurately from the graph. If interpolation between the grade curves is necessary, the inaccuracies are compounded. It is for these reasons that a more comprehensive description of the speed-distance curves is desirable. An alternative approach involving the application of speed-distance tables is presented in the Appendix.

Capacity Analysis

Studies reported in 1957 (4) show that in level terrain the average dual-tired vehicle is equivalent, in a capacity sense, to 2.5 passenger cars on 2-lane highways and 2 passenger cars on multilane highways. From these studies, the AASHO Policy incorporates a "passenger-car equivalent" chart (also shown in the Appendix) for determining the space relationship for 2-lane highways under conditions producing speed differentials greater than those found on nearly level highways. In addition, it was assumed that on multilane highways passenger-car equivalents are approximately 80 percent of the values for 2-lane highways.

The 1957 findings (4) indicate that these passenger-car equivalents change very little with the percentage of trucks in the traffic stream. Rather, it is the difference between truck speeds and passenger car speeds on grades that causes trucks to reduce the capacity of a highway.

To use the AASHO table for passenger-car equivalents in determining the design capacity of a particular grade requires that the average running speed of the design vehicle be estimated. The AASHO Policy provides data for this purpose (1, Table V-4). The data given in Table V-4 actually do not represent the performance of the same heavy truck for which data are shown in Figure 1. The only reason for switching to a different design vehicle at this point in the climbing-lane design procedure appears to be that this table had already been published (4), and construction of a more appropriate one would have been somewhat laborious. [One would have to compute average speeds by using the relationships developed by Huff and Scrivner (3).]

The AASHO Policy states that the design capacity of a highway grade can be determined from the following equation:

$$\text{Design capacity} = \frac{100 \times \text{capacity with no trucks}}{100 + T(j - 1)}$$

where

design capacity = vehicles per hour;
T = percentage of trucks in the design traffic stream; and
j = passenger-car equivalent (for 2-lane highways, a weighted equivalent accounting for both directions of travel).

"Capacity with no trucks" is taken from AASHO Policy (1, Tables II-8 through II-10) for 2-lane highways as appropriate for the given condition of average running speed, lane width, and sight-distance restrictions. The calculated design capacity in this case is for both directions of traffic. In computations of grade capacities of multilane highways, lane capacity values given in Table II-12 are used. Capacity, in this case, is determined in the uphill direction only, based on the percentage of trucks in that direction during peak hours.

One dubious procedure is in computing the weighted passenger-car equivalent for 2-lane highways. Here the AASHO Policy assumes that the passenger-car equivalent on the downgrade is 2.5, the same as used for level sections. A more realistic approach would consider what effect the preceding upgrade has had on those trucks traveling down the grade in question.

Treatment of Vertical Curves

The speed-distance relationships described by AASHO's grade performance curves are for operation on tangent grade segments. Because a minor speed change can have a substantial effect on the lengths of climbing lanes designed with speed criteria, some attempt should be made to predict truck performance on vertical parabolic curves as well as on connecting grades. Simply using the gradients of the tangents to the VPI is to be discouraged because of the large amount of uncertainty introduced by ignoring the true profile, especially for sizable algebraic differences in percentage of grade and for moderately long curves.

There are at least 2 basic approaches to the accommodation of vertical curves in a grade performance analysis. The one prescribed by AASHO refers to the following 4 types of crest and sag curves (Fig. 3):

> Where the condition involves vertical curves of types II and IV . . . and the algebraic difference in grades is not too great, the measurement of critical length of grade may be made between the VPI points Where vertical curves of types I and III are involved, particularly where the algebraic difference in grades is appreciable, about one-quarter of the vertical curve length may be considered as part of the grade under consideration.

This is a rather inprecise approach. Because it would take substantial numerical synthesis to arrive at quantitative replacements for "not too great" and "appreciable," AASHO chose to leave the issue unresolved. The engineer must depend on judgment and judgment alone. Standard quantitative procedures should be employed wherever they can be reasonably established.

Another approach, which is based on a logical examination of the problem and a potential for further refinement, can be developed. The point of a vertical curve of

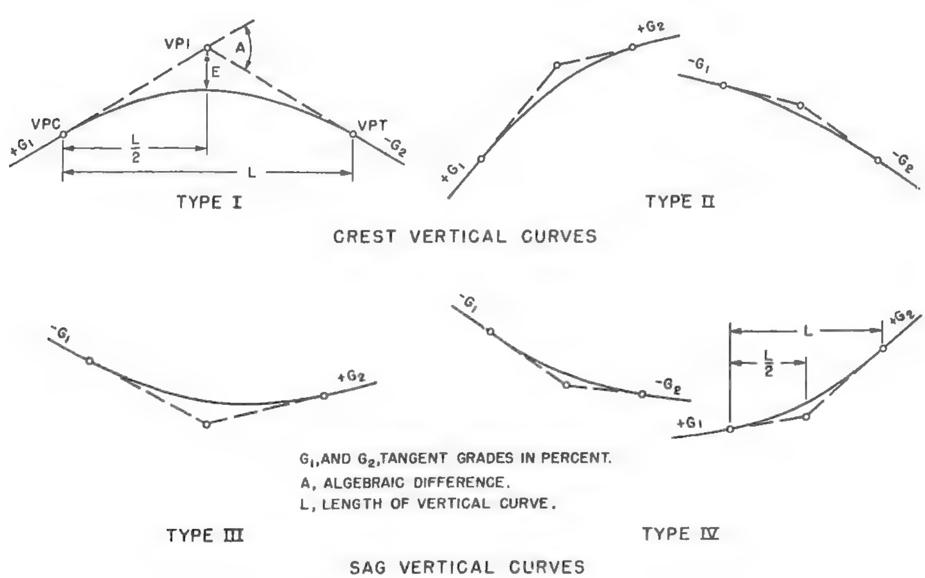

Figure 3. Types of vertical curves.

greatest consequence to the problem at hand is the highest point (turning point) on a crest vertical curve. The gradients from the ends of the curve to the turning point are exactly one-half of the gradients on the tangents. Hence, an easily computed initial approximation to the profile traveled by the design vehicle consists of 2 chords meeting at the turning point. This stage of reduction should be regarded as a minimum in any grade performance analysis. A brief examination of a typical profile shows that even this simple approximation is more accurate than the AASHO suggestion that "about one-quarter of the vertical curve length may be considered as part of the grade under consideration."

If greater accuracy is subsequently deemed desirable, the vertical curve could be examined by smaller chord lengths, perhaps 100-ft intervals. This standard chord-gradient method (5) will then define a more precise level of chord approximation. Odd percentages of grade will prove much less difficult to handle by using the grade performance tables (explained in the Appendix) than by using the AASHO curves; table resolution is one-half percent, and interpolation to the nearest tenth percent can be done with some confidence.

DESIGN PROCEDURES

It is important to note that the AASHO Policy heavily stresses the concept of "critical length of grade" for analysis of upgrade operation but fails to place a correspondingly adequate emphasis on a critical length of grade for speed recovery and lane change beyond the summit. The most prevalent current practice warrants the initiation of a climbing lane at that point on the grade where the design vehicle's speed has been reduced by 15 mph. Likewise, it is a frequent practice to end the climbing lane at that point where the design vehicle regains that speed for which the lane was initiated. This practice, for many profiles, permits the termination of the extra lane shortly over the crest of the hill. In this regard, the AASHO Policy suggests ending a climbing lane no sooner than 200 ft beyond the point where passing sight distance becomes available. This minimum is precariously short from a safety standpoint. Inadequate operational sight distance might exist, especially for automobile drivers who choose to use the auxiliary lane because they are traveling somewhat slower than other vehicles in the traffic stream.

Professional truck drivers have voiced objection to the termination of climbing lanes at a point where their vehicles have not regained sufficient speed to safely reenter the through lane. A merging situation could be especially hazardous. Passenger car drivers report being forced into rapid lane changes where climbing lanes end prematurely (6).

SUMMARY OF DESIGN RECOMMENDATIONS

Each highway alignment is worthy of a thorough analysis in which an examination is made of the grade performance of a design vehicle, the relation between future traffic volumes and capacity, the potential accident involvement due to extreme speed differentials in the traffic stream, and the availability of passing and weaving sight distances. The remaining portion of this report deals with the recommendations for achieving climbing-lane designs that are relatively safe and efficient.

Design Warrants

For determining what constitutes a critical length of grade, the adoption of a 10-mph maximum speed-reduction criterion is strongly recommended. Substituting this criterion for the currently used 15-mph speed reduction of the design vehicle reduces the expected truck accident involvement rate on grades by nearly 60 percent.

Under present traffic conditions, the volume warrants of 120 percent and 130 percent of design capacity for 2-lane and multilane highways respectively are excessive. It does not appear logical to deliberately allow a hazardous bottleneck to be built into a highway. It is, therefore, recommended that the capacity overrun criteria be eliminated. In place of these criteria, it is recommended that climbing lanes be warranted on critical lengths of grade if the DHV equals or exceeds the design capacity of the grade.

Commencement and Termination of Climbing Lanes

The basic tack to follow in determining the location to commence a climbing lane consists of a more empathic examination of the speed-differential, accident-involvement relationship. The 10-mph speed-reduction criterion is desirable, but it is not necessarily the least reduction that should be considered. An examination of the speed-distance tables given in the Appendix shows that speed reductions of less than 10-mph do not substantially increase the required climbing-lane length for the steeper grades. For example, in decelerating from 42 mph to 37 mph on a 6 percent upgrade, the design truck travels only 230 ft. The cost of this small addition of length to the climbing lane, using a 5-mph rather than a 10-mph speed-reduction basis, would be more than offset by the concomitant reduction in accident costs. It should be kept in mind that the roadway earthwork will serve indefinitely, and accidents could occur any day of that indefinite period.

Considerations similar to these are recommended for the termination of climbing lanes. In addition, consideration should be given to available sight distance. For all grade designs, the pavement surface at the end of the climbing lane should be visible at the following distances (7):

Sight Distance (ft, height of object is zero)	Design Speed (mph)
600	50
750	60
900	70
1,100	80

These recommended sight distances would at least allow those passenger-car drivers who use the auxiliary lane (because of a speed somewhat lower than that of other vehicles in the traffic stream) to come to an emergency stop if vehicles in the through lane prevent a lane change before the end of the auxiliary lane.

In terrain that dictates consecutive climbing lanes at short intervals, it is recommended that serious consideration be given to joining the separate climbing lanes to form one continuous lane. This would eliminate the somewhat hazardous situation of forcing reentry of the truck into the normal flow of traffic and then, in a short distance, of removing it again. Truck drivers will tend to avoid this type of operation, and impatient automobile drivers may usurp the climbing lane for high-speed passing. This behavior would be extremely dangerous where sight distance is restricted.

Analysis Procedures

It is recommended that the speed-distance tables given in the Appendix be substituted for the AASHO speed-distance curves. These tables greatly enhance the accuracy of grade performance analysis.

In the analysis of grade performance on vertical curves, it is recommended that the chord-gradient method using a 100-ft chord length be employed. This allows a greater precision than the procedures now commonly utilized.

Design Elements

The minimum width of auxiliary lanes should be equal to that of the continuous lanes. In no case should this be less than 11 ft. Truck drivers must not be discouraged from proper use of a climbing lane by a width inferior to that of the other traffic lanes. If less than full-width shoulders are planned, a 13-ft-wide climbing lane is desirable. The speed differential between vehicles traveling the same direction in adjacent lanes also provides some justification for a wider-than-normal lane.

A full-width (8 to 10 ft) paved shoulder is advisable, and 6 ft should be a desired minimum. Without a wide paved shoulder, a stalled vehicle must occupy part of the climbing lane, and motorists approaching from the rear are forced to change lanes or remain apprehensive as to whether a vehicle in the right lane might change lanes in

front of them. Diagnostic field studies (6) have revealed that these possibilities concern drivers, particularly over the crest of the vertical curve or where there is a chance that a slow truck in the climbing lane may have to swerve into the high-speed lane to bypass a stalled vehicle. A full-width shoulder is particularly recommended immediately downstream from the end of a climbing lane to allow for emergency maneuvers.

The desirable length of climbing lane tapers is found by the expression

$$L = S \times W$$

where L is the taper length in feet, S is the design speed or 85th percentile speed in mph, and W is the climbing lane width in feet. This recommendation is in accordance with the Manual on Uniform Traffic Control Devices (8) for 2- to 3-lane pavement width transitions.

For pavement cross slope, the AASHO Policy recommendation should apply. The cross slope of the truck lane should usually be a continuation of that on the adjacent traffic lane.

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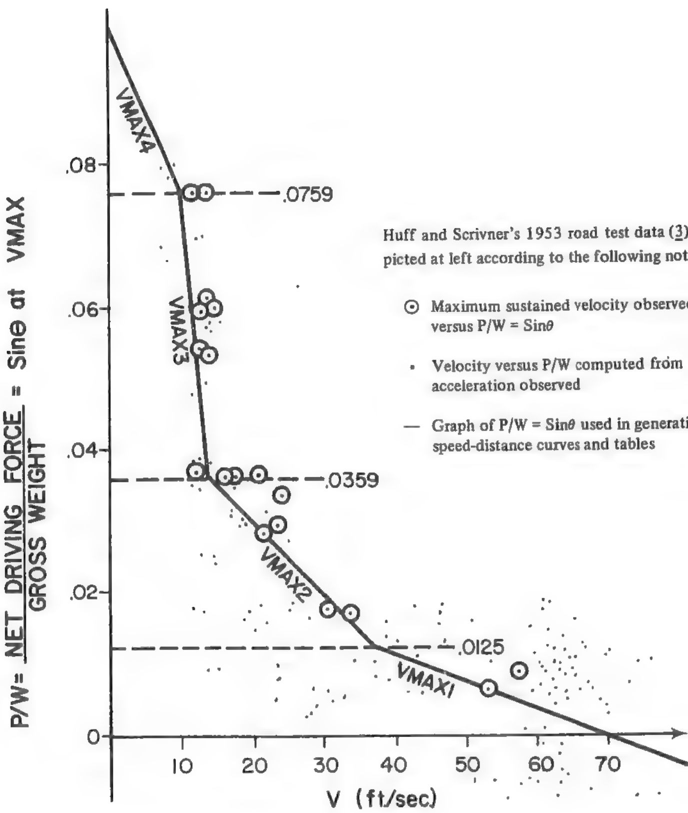
APPENDIX

SPEED-DISTANCE TABLES

Development

The procedural difficulties inherent in grade performance curves used in design are largely alleviated through the application of tables giving the same kind of information in a more convenient format. Stimpson (9) has written computer programs for generating speed-distance tables for the Huff-Scrivner design truck used in the AASHO Policy. The development is based on an empirical graph (Fig. 4) and the following speed-distance equation (2):

$$d = \frac{\cos\theta}{g} \frac{V^2 - V_o^2}{a(V + V_o) - 2(\sin\theta - b)}$$



Huff and Scrivner's 1953 road test data (3) are depicted at left according to the following notation:

⊙ Maximum sustained velocity observed versus P/W = Sinθ

• Velocity versus P/W computed from acceleration observed

— Graph of P/W = Sinθ used in generating speed-distance curves and tables

To obtain the values "a" and "b" for use in the general motion equation, the following equations for the VMAX line segments were written:

V (fps)	Line Segment	Sinθ = function of V	"a"	"b"
0.00 – 10.00	VMAX4	–.00241V + .1000	–.00241	.1000
10.00 – 13.75	VMAX3	–.01067V + .1826	–.01067	.1826
13.75 – 36.54	VMAX2	–.001027V + .0500	–.001027	.0500
36.54 – 68.93	VMAX1	–.000374V + .0262	–.000374	.0262

Note that "b" is the P/W intercept of the extrapolated VMAX line segment, and "a" is the slope of the line segment.

Figure 4. Equations for maximum sustained velocity.

where

- d = horizontal component of displacement on grade, ft;
- θ = grade angle, deg;
- g = acceleration due to gravity, 32.1725 ft/sec^2;
- V = velocity of design truck at distance d from origin, ft/sec;
- V_o = velocity of design truck at origin, ft/sec; and
- a, b = constants applicable in velocity ranges shown in Figure 4.

To determine total displacements where the velocity change involves more than one VMAX line segment (Fig. 4), the distance d must be computed over each interval and added. The origin referred to is the truck location at the beginning of the speed interval to which the equation is being applied. A long series of speed-distance expressions for the various ranges of possible velocity change were subsequently written, and each of the speed-distance tables were constructed via a different Fortran IV-G computer program utilizing the same basic algorithm.

Application

The implementation of the speed-distance tables (Figs. 5, 6, and 7) in design is fairly straightforward and efficient. It is considerably more accurate than the graphical approach. Before any speed-reduction criteria can be applied, the actual performance of the design truck must be traced through the tables. The same example problem solved graphically in Huff and Scrivner's report (3) is shown in Figure 8. A solution to this problem involving the application of tables shown in Figures 5 and 6 is discussed below.

When a proposed profile has been established, the performance of the typical heavy truck must be determined as it moves from one grade to another. Entering on a

MPH	PERCENT GRADE 0.0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0
46	0	730	340	220	160	130	110	90	80	70	60	60	50	50	40	40	40
45	0	1540	680	440	320	260	210	180	160	140	130	110	100	100	90	80	80
44	0	2450	1050	670	490	380	320	270	240	210	190	170	160	140	130	120	120
43	0	3480	1420	890	650	510	420	360	310	280	250	230	210	190	180	160	150
42	0	4680	1820	1130	820	640	530	450	390	340	310	280	260	240	220	200	190
41	0	6070	2230	1370	990	770	630	540	470	410	370	330	300	280	260	240	230
40	0	7730	2670	1610	1150	900	740	620	540	480	430	390	350	330	300	280	260
39	0	9740	3130	1860	1330	1030	840	710	620	540	490	440	400	370	340	320	300
38	0	0	3610	2120	1500	1160	940	800	690	610	540	490	450	410	380	350	330
37	0	0	4120	2380	1670	1290	1050	380	760	670	600	540	490	450	420	390	370
36	0	0	4670	2650	1850	1420	1150	970	840	740	660	590	540	500	460	430	400
35	0	0	5250	2920	2020	1550	1250	1050	910	800	710	640	580	540	500	460	430
34	0	0	5880	3200	2200	1680	1350	1140	980	860	770	690	630	580	530	500	460
33	0	0	6550	3490	2380	1810	1460	1220	1050	920	820	740	670	620	570	530	490
32	0	0	7260	3790	2570	1940	1560	1300	1120	980	870	790	710	660	610	560	530
31	0	0	8070	4100	2750	2070	1660	1380	1190	1040	920	830	760	690	640	590	560
30	0	0	9940	4420	2940	2200	1760	1460	1250	1100	980	880	800	730	670	630	580
29	0	0	0	4750	3130	2330	1860	1540	1320	1150	1030	920	840	770	710	660	610
28	0	0	0	5100	3320	2460	1960	1620	1390	1210	1070	970	880	800	740	690	640
27	0	0	0	5450	3520	2590	2060	1700	1450	1270	1120	1010	920	840	770	720	670
26	0	0	0	5830	3710	2730	2150	1780	1520	1320	1170	1050	950	870	800	750	690
25	0	0	0	6210	3920	2860	2250	1860	1580	1370	1220	1090	990	910	830	770	720
24	0	0	0	7060	4150	3000	2350	1930	1640	1430	1260	1130	1020	940	860	800	750
23	0	0	0	0	4440	3150	2450	2010	1700	1480	1310	1170	1060	970	890	830	770
22	0	0	0	0	4790	3310	2560	2090	1770	1530	1350	1210	1090	1000	920	850	790
21	0	0	0	0	5250	3500	2670	2170	1830	1580	1390	1250	1130	1030	950	880	820
20	0	0	0	0	5860	3710	2790	2260	1900	1640	1440	1290	1160	1060	970	900	840
19	0	0	0	0	0	3950	2920	2340	1960	1690	1480	1320	1190	1090	1000	930	860
18	0	0	0	0	0	4230	3070	2440	2030	1740	1530	1360	1230	1120	1030	950	880
17	0	0	0	0	0	4560	3220	2530	2100	1800	1570	1400	1260	1150	1050	970	900
16	0	0	0	0	0	0	3390	2640	2170	1850	1620	1440	1290	1170	1080	990	920
15	0	0	0	0	0	0	3580	2740	2250	1910	1660	1470	1320	1200	1100	1020	940
14	0	0	0	0	0	0	3790	2860	2320	1960	1710	1510	1350	1230	1120	1040	960
13	0	0	0	0	0	0	0	2980	2400	2020	1750	1550	1380	1260	1150	1060	980
12	0	0	0	0	0	0	0	3110	2480	2080	1790	1580	1410	1280	1170	1080	1000
11	0	0	0	0	0	0	0	3250	2570	2140	1840	1620	1440	1310	1190	1100	1020
10	0	0	0	0	0	0	0	3410	2650	2200	1880	1650	1470	1330	1210	1120	1030
9	0	0	0	0	0	0	0	0	0	2270	1930	1690	1500	1350	1230	1130	1050
8	0	0	0	0	0	0	0	0	0	0	0	0	1550	1390	1260	1150	1070
7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1190	1090
6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1250
MSS	47.0	38.2	29.3	23.3	19.9	16.6	13.3	10.0	9.1	8.8	8.5	8.2	7.8	7.5	7.2	6.9	5.7
	0	11810	9570	8340	5910	4710	3950	3410	2790	2310	1990	1750	1560	1420	1300	1200	1340

10-mph reduction

Start 1,670' up 2% grade

2,500

1,090

Add 1,100'

2,190

15.75

α

Note: "MSS" is the "maximum sustained speed"; distances shown below "MSS" row are those at which the crawl speed is first established.

Figure 5. Speed-distance table for heavy truck deceleration upgrade (horizontal distance in feet).

MPH	0.0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0
	PERCENT GRADE																
1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	10	10	10
3	0	0	0	0	0	0	0	10	10	10	10	10	10	10	10	20	20
4	10	10	10	10	10	10	10	10	10	10	10	10	20	20	20	30	40
5	10	10	10	10	10	10	10	10	20	20	20	20	30	30	40	50	70
6	10	10	20	20	20	20	20	20	20	30	30	30	40	50	60	80	0
7	20	20	20	20	20	30	30	30	30	40	40	50	60	80	100	0	0
8	30	30	30	30	40	40	40	50	50	60	80	100	0	0	0	0	0
9	40	40	40	50	50	60	70	80	90	0	0	0	0	0	0	0	0
10	50	60	70	70	90	110	150	0	0	0	0	0	0	0	0	0	0
11	70	80	90	110	140	180	320	0	0	0	0	0	0	0	0	0	0
12	100	110	130	150	200	280	560	0	0	0	0	0	0	0	0	0	0
13	120	140	170	200	270	400	940	0	0	0	0	0	0	0	0	0	0
14	150	180	210	260	350	560	0	0	0	0	0	0	0	0	0	0	0
15	190	220	260	330	450	760	0	0	0	0	0	0	0	0	0	0	0
16	220	260	320	410	590	1020	0	0	0	0	0	0	0	0	0	0	0
17	270	310	380	500	720	0	0	0	0	0	0	0	0	0	0	0	0
18	310	370	460	600	900	0	0	0	0	0	0	0	0	0	0	0	0
19	360	430	540	720	1120	0	0	0	0	0	0	0	0	0	0	0	0
20	420	500	630	870	0	0	0	0	0	0	0	0	0	0	0	0	0
21	480	580	740	1030	0	0	0	0	0	0	0	0	0	0	0	0	0
22	540	670	860	1220	0	0	0	0	0	0	0	0	0	0	0	0	0
23	620	760	990	1440	0	0	0	0	0	0	0	0	0	0	0	0	0
24	700	870	1150	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	790	990	1360	0	0	0	0	0	0	0	0	0	0	0	0	0	0
26	930	1230	2130	0	0	0	0	0	0	0	0	0	0	0	0	0	0
27	1080	1490	3140	0	0	0	0	0	0	0	0	0	0	0	0	0	0
28	1250	1790	4510	0	0	0	0	0	0	0	0	0	0	0	0	0	0
29	1420	2120	6460	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	1620	2490	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31	1820	2910	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
32	2050	3380	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
33	2290	3910	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
34	2560	4510	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
35	2840	5200	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	3150	5960	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
37	3490	6880	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
38	3850	7920	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
39	4240	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
40	4670	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
41	5130	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
42	5640	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
43	6190	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
44	6790	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
45	7440	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
46	8160	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
MSS	47.0	38.2	29.3	23.3	19.5	16.6	13.3	10.0	9.1	8.6	8.5	8.2	7.8	7.5	7.2	6.9	5.7
	8950	8100	7290	1510	1370	1230	1090	1000	100	100	110	110	120	130	150	180	110

a 15.75

305

Add 700'

630

1,005

End (3490 - 630) = 2860' along 0% grade

10-mph reduction

Figure 6. Speed-distance table for heavy truck acceleration upgrade (horizontal distance in feet).

MPH	0.0	-0.5	-1.0	-1.5	-2.0	-2.5	-3.0	-3.5	-4.0	-4.5	-5.0	-5.5	-6.0	-6.5	-7.0	-7.5	-8.0
	PERCENT GRADE																
1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	10	10	10	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	0
6	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
7	20	20	20	20	20	10	10	10	10	10	10	10	10	10	10	10	10
8	30	20	20	20	20	20	20	20	20	20	20	20	20	10	10	10	10
9	40	30	30	30	30	30	30	20	20	20	20	20	20	20	20	20	20
10	50	50	50	40	40	40	40	30	30	30	30	30	30	30	20	20	20
11	70	70	60	60	50	50	50	40	40	40	40	40	30	30	30	30	30
12	100	90	80	70	70	60	60	60	50	50	50	40	40	40	40	40	40
13	120	110	100	90	80	80	70	70	60	60	60	50	50	50	50	50	40
14	150	140	120	110	100	90	90	80	80	70	70	60	60	60	60	50	50
15	190	160	150	130	120	110	100	100	90	90	80	80	70	70	70	60	60
16	220	200	170	160	140	130	120	110	110	100	90	90	80	80	80	70	70
17	270	230	200	180	170	150	140	130	120	110	110	100	100	90	90	80	80
18	310	270	240	210	190	180	160	150	140	130	120	120	110	110	100	100	90
19	360	310	270	240	220	200	180	170	160	150	140	130	130	120	110	110	100
20	420	360	310	280	250	230	210	190	180	170	160	150	140	130	130	120	120
21	480	410	350	310	280	260	240	220	200	190	180	170	160	150	140	140	130
22	540	460	400	350	320	290	260	240	230	210	200	190	180	170	160	150	140
23	620	520	450	400	360	320	290	270	250	230	220	210	190	180	170	170	160
24	700	590	500	440	400	360	330	300	280	260	240	230	210	200	190	180	170
25	790	660	560	490	440	400	360	330	310	290	270	250	240	220	210	200	190
26	930	760	640	560	490	440	400	370	340	320	290	280	260	250	230	220	210
27	1080	860	720	620	550	490	440	410	370	350	320	300	280	270	250	240	230
28	1250	980	810	690	610	540	490	450	410	380	350	330	310	290	280	260	250
29	1420	1100	900	770	670	600	540	490	450	410	380	360	340	320	300	280	270
30	1620	1230	1000	850	740	650	560	530	490	450	420	390	370	340	320	310	290
31	1820	1370	1100	930	800	710	640	580	530	490	450	420	390	370	350	330	320
32	2050	1510	1210	1020	880	770	690	620	570	530	490	450	420	400	360	360	340
33	2290	1670	1330	1110	950	840	750	670	620	570	520	490	460	430	400	380	360
34	2560	1840	1450	1200	1030	900	800	730	660	610	560	520	490	460	430	410	390
35	2840	2020	1560	1300	1110	970	870	780	710	650	600	560	520	490	460	440	420
36	3150	2210	1720	1410	1200	1050	930	840	760	700	640	600	560	520	490	470	440
37	3490	2410	1860	1520	1290	1120	1000	890	810	740	690	640	600	560	530	500	470
38	3850	2620	2010	1640	1390	1200	1060	950	870	790	730	680	630	590	560	530	500
39	4240	2850	2170	1760	1480	1290	1140	1020	920	840	780	720	670	630	590	560	530
40	4670	3090	2340	1890	1590	1370	1210	1080	980	900	820	760	710	670	630	590	560
41	5130	3350	2510	2020	1700	1460	1290	1150	1040	950	870	810	750	710	660	630	590
42	5640	3620	2700	2160	1810	1560	1370	1220	1100	1010	920	860	800	750	700	660	630
43	6190	3910	2890	2310	1920	1650	1450	1290	1170	1060	980	900	840	790	740	700	660
44	6790	4220	3100	2460	2050	1750	1540	1370	1230	1120	1030	950	890	830	780	730	690
45	7440	4550	3310	2620	2170	1860	1630	1450	1300	1150	1090	1000	930	870	820	770	730
46	8160	4900	3540	2790	2300	1970	1720	1530	1370	1250	1140	1060	980	920	860	810	770
47	8950	5270	3780	2960	2440	2080	1610	1610	1450	1310	1200	1110	1030	960	900	850	800

Figure 7. Speed-distance table for heavy truck acceleration downgrade (horizontal distance in feet).

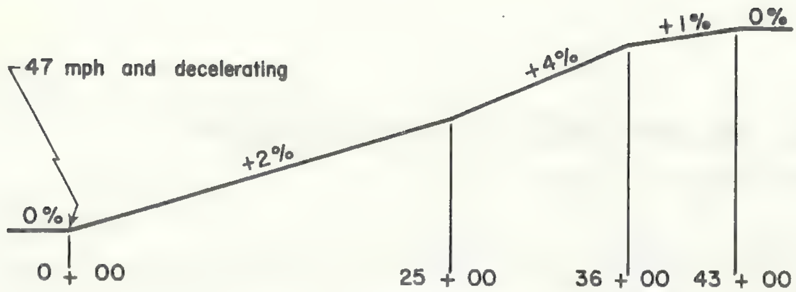


Figure 8. Example grade problem.

2 percent grade, the truck decelerates for a distance of 2,500 ft before reaching an even steeper grade (4 percent). Linear interpolation in the table for a 2,500-ft horizontal displacement establishes a "distance number" in the 4 percent column. The length of the 4 percent grade (1,100 ft) is added to the first distance number, and a vertical line is dropped to the second distance number. These are called distance numbers to emphasize the fact that they are only differences. Absolute magnitude is important only on the first and last grades in the series being studied. Because the truck goes from a 4 percent grade to a 1 percent grade, acceleration begins (though not at the true summit), and we must transfer to another table. This is done essentially as in the graphical method: Determine the speed appropriate at the second distance number and enter the next table at the same speed. One should interpolate for the speed to at least the nearest 0.1 mph.

TABLE 1

CLIMBING-LANE LENGTH DETERMINATION FOR VARIOUS SPEED-REDUCTION DESIGN CRITERIA

Speed Reduction	Speed (mph)	Engineering Station: Begin Full Climbing Lane	Engineering Station: End Full Climbing Lane	Length (ft)
0	47	0 + 00	126 + 20	12,620
5	42	8 + 20	93 + 10	8,490
10	37	16 + 70	71 + 60	5,490
15	32	25 + 30	57 + 20	3,190
20	27	28 + 60	47 + 50	1,890
25	22	31 + 80	41 + 60	980
30	17	35 + 10	36 + 80	170
Max.	15.75	36 + 00	36 + 00	0

A third distance number is interpolated in the 1 percent column on the table for acceleration upgrade at the speed used in the transfer. A fourth distance number is found in a fashion similar to that used for finding the second number. The last distance number is interpolated in the 0 percent column by proportions determined by the fourth number. If this is the last grade in the declared study section, acceleration will continue until a speed criterion is reached or another grade is encountered, whichever comes first. If the second contingency occurs, the study section was improperly defined and computations continue.

A performance table such as that shown in Figure 8 can be rapidly compiled by use of the grade performance tables and simple subtraction. The right column represents but one of many possible approaches to determining climbing-lane length. Depending on the accepted design minimums and various economic factors as they relate to the proposed profile, a speed-reduction criterion applied to locate one end of a climbing lane might be different from that used for the other end. Because the actual design truck performance is independent of extraneous design standards, a number of feasible alternatives can be described by taking diagonal combinations of engineering stations specifying the beginning and the end of a full-width climbing lane. The alternative designs can then be analyzed and compared on a benefit-cost basis. The data given in Table 1 illustrate the determination of climbing-lane length for various speed-reduction design criteria.

OPTIMAL GEOMETRIC DESIGN DECISIONS FOR HIGHWAY SAFETY

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The continuing high accident rate on the highways and the associated social and economic cost have made improvement in highway safety one of the high priority objectives of agencies responsible for road investment and management. Our knowledge of factors contributing to highway accidents is not complete, nor is our knowledge of the value of accident reductions. In dealing with these problems, the highway engineer must make decisions with respect to model or models to use or to develop ways to deal with the uncertainty of his predictions and evaluations, and changes to implement. This paper presents a framework that was developed to deal with these and related problems and describes its pilot applicator to a rural spot-improvement problem in the Midwest.

•THE CONTINUING high accident rate on the highways and the associated social and economic cost have made improvement in highway safety one of the high priority objectives of road investment and management agencies at all levels of government (23). As a result of the importance of this problem, much effort has been expended to increase our knowledge of those factors affecting highway safety and to provide the information needed for making sound decisions about programs designed to improve safety performance. Research has substantially increased our knowledge of the relationships between traffic safety levels and variables potentially or actually within the control of highway management, including the vehicle (25), roadway, driver, and environment. This knowledge, however, is not complete (9, 14). In some cases, the relationships are only vaguely known, while in others the models resulting from the research predict different levels of effectiveness for identical changes or actions in the same situation (3). Similarly, the costs associated with a particular accident level and composition are not known precisely, and different results are obtained in different studies (4). Thus, there is considerable uncertainty associated with the prediction of accident levels resulting from actions or decisions intended to increase safety, and there is uncertainty in the value of such accident rate changes (19).

These conditions lead to many problems for the highway engineer who makes decisions about means to improve safety, whether in designing a new highway or in modifying an existing one. First, he must decide on a methodology to use for predicting accident rates, including the possibility of using existing predictive relationships or of developing his own. Second, he must select or devise a means for evaluating the alternatives. Third, he must decide on how to modify, if at all, the results of safety level predictions and the associated evaluations in order to make them more realistic by using his own and his colleagues' experience and judgment. Finally, he should employ a methodology that explicitly considers information on the extent to which predictions and evaluation may be in error.

The research reported in this paper was directed toward the development of a decision-making framework that explicitly deals with these problems facing the highway engineer.

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The research concentrated on the relationships between geometric design and accident levels, although the methodology could be applied to other factors such as vehicle standards and enforcement. A description is given of the informational base on which geometric design decisions must be made with respect to traffic safety. This provides the basis for the decision-making framework presented. Finally, the results of an application to a high accident problem at a rural midwestern location are presented.

PREDICTION AND EVALUATION

Prediction

A substantial amount of work has been done on the relationship of accident rates and one individual geometric design feature such as shoulder width, lane width, and curves (1, 5, 7). Ideally, one desires a highly inclusive model that simultaneously relates accident rate to highway geometrics, vehicle characteristics, driver behavior, traffic characteristics, and environment including weather and pavement conditions. Obviously, these relationships are highly complex and interactive and are only partly understood at this time (16, 19).

However, some equations have been structured that attempt to assess the relationship of certain combinations of geometric design components to accident rates (10, 11). Typical of these are those by Kihlberg and Tharp (17) and those by Dart and Mann (21). The Kihlberg and Tharp equations use a logarithmic relationship to test whether significant differences in accident rates exist between sections that have the same ADT but possess a variety of combinations of curves, grade, structure, and intersection and sections that are geometrically pure but of the same general highway type. These relationships are developed as follows (17):

$$\log \overline{A} = a + b_1 \log \overline{T} + b_2 \log^2 \overline{T} \tag{1}$$

where

$\overline{T}$ = mean ADT; and
$\overline{A}$ = mean annual number of accidents on a 0.3-mile section.

When, from original regression analysis runs, the a, b_1, and b_2 are found, the smooth curve is computed as follows:

$$y_i = a + b_1\hat{x}_{1i} + b_2\hat{x}_{2i} \tag{2}$$

where

$\hat{x}_{1i} = \frac{1}{2}(\log T_{Li} + \log T_{Ui})$;
T_{Li} = lower ADT limit of the ith class;
T_{Ui} = upper ADT limit of the ith class;
$\hat{x}_{2i} = \hat{x}_{1i}^2$; and
antilog $y_i = \overline{A}_i$ = predicted number of accidents in the ith ADT class.

Likewise, typical of the Mann and Dart regression results is the result found for total accidents (21):

$$\begin{aligned} Y_1 = \text{total accidents/100 mvm} &= 41.32 - 1.23X_2 - 0.54X_3 - 0.67X_6 + 0.03X_2X_3 \\ &+ 0.02X_3X_6 - 0.0009X_3X_9 + 0.034X_3X_{11} \\ &- 0.2X_4X_{11} + 0.009X_5X_9 \end{aligned} \tag{3}$$

where

X_1 = number of lanes, total for both directions;
X_2 = percentage of trucks;
X_3 = traffic volume-capacity ratio for operation at level of service B;
X_4 = lane width, ft;

X_5 = shoulder width, ft;
X_6 = cross slope, in./ft;
X_7 = percentage of continuous obstructions (percentage of highway length);
X_8 = marginal obstructions per mile;
X_9 = horizontal alignment, percentage of length in excess of 3 deg;
X_{10} = vertical alignment, percentage of length in excess of 3 percent; and
X_{11} = traffic access points per mile.

Percentage of continuous obstruction is defined as the percentage of the total length of a highway section that has some roadside feature or obstacle that runs for more than a few feet on either or both sides of the roadway. Such a feature would be a deep roadside ditch or steep side slope that presents an obstacle to a vehicle safely leaving the roadway in an emergency at posted highway speeds.

Marginal obstructions per mile are defined as the total number of discrete objects on both sides within the cleared right-of-way per mile of a highway section. Those objects may be a driveway embankment culvert, roadway culvert, headwall, tree, or telephone pole. This term is not to be confused with the term used in capacity analysis to refer to marginal obstruction within 6 ft of the pavement edge.

Several problems exist if one considers using either of these typical predictors.

1. In almost all instances, when used on the same rural section, the resulting accident rates predicted by one model are substantially different from those predicted by the other. This raises the problem of choice of predictive relationship and the question of whether it is better to develop a relationship specifically for the region and sites under consideration.
2. Each of the results is based on data from local areas, and the question of transferability of results to other problem areas exists. This further reinforces the problems mentioned earlier.
3. An obvious problem in the quality of the relationships is the omission of certain highly important variables, which are often difficult to measure. The most critical of these in highway safety are the driver and his responses and the vehicle type and condition.
4. The engineers involved normally have a substantial background of experience and judgment to draw from when they view the results of predictive models, such as those described here. Any overall evaluation methodology using predictive relationships should utilize this judgment.

Evaluation

In addition to prediction, certain evaluation techniques have been developed for studying accident phenomena associated with geometric design. These are based on a variety of criteria for decision as to whether a site is considered hazardous. These criteria, thoroughly discussed by Jorgensen (16) are stated briefly as follows: number method, rate method, number-rate method, and rate quality control method (8, 15).

All of them can induce some distortion, depending on the position taken by the analyst with respect to number of accidents and exposure. Different rates, numbers, or levels of confidence (in the case of the rate quality control method) can result in differing groups of sites being declared as hazardous.

The current evaluation techniques make recommendations on the basis of appropriate knowledge of capital and maintenance costs, accident costs, time, delay, and comfort and convenience costs (2, 18). The following commonly used techniques are familiar to the highway engineering community (16): total minimum annual cost, benefit-cost ratio, rate of return, net benefits, and cost-effectiveness analysis.

Appropriate costs for various types of accidents have recently been studied in detail (30, 31). In addition, monetary relationships between fatal and severe-injury accidents and an appropriate equivalent number of property-damage accidents are available (26).

The evaluation of safety consequences in dollar measurements alone has been a point of contention for some time. Various groups affected by accident phenomena attach different values to the consequences, such as those involving injury to the occupants,

roadside property damage, and the resulting congestion and inefficiency of the highway system. Therefore, the solutions may be dependent on, or altered by, the viewpoints of groups included. It is important to consider as wide a variety of viewpoints as possible in any evaluation process conducted by officials of a public agency. The evaluation schemes may possibly be broadened to make use of any or all of the economic evaluation techniques given earlier and also may incorporate as many points of view as possible.

Summary

The pertinent aspects of the prediction and evaluation phases of traffic safety problems are summarized as follows:

1. In the assessment of accident rates, the wide variety of quality of relationships and data bases in relation to geometric design variables often results in substantially different predictions of rates for the same highway section.
2. The judgment and insight of the designer can be used to modify results of models to yield a more accurate prediction for local situations that have great influence on accident rates. Well-documented, typical situations of this type include local sections with very poor driver behavior despite reasonably adequate design, such as those where there is a large incidence of drunken drivers or drivers in certain age groups who operate vehicles carelessly and recklessly.
3. It appears possible to incorporate the usual engineering evaluation techniques into a decision-making framework that allows the public agency to consider as many points of view as possible.
4. There is often uncertainty associated with the costs for alleviating the situation. In addition, the benefits accruing from design modification are subject to even greater uncertainty. These aspects of uncertainty should be incorporated into the decision-making framework.

The next section discusses an approach based on decision theory for dealing with these aspects of prediction and evaluation of alternative design modifications to improve safety.

DECISION THEORY APPROACH

Decision theory is a managerial tool that has been developed for dealing with problems similar to those encountered in this field of highway engineering; it has been widely used in dealing with business problems (27). To understand the adaptions for and applications to highway safety investment decisions, one must first know the major characteristics of the theory.

Basic Problem Structure

The basic approach is to break large, complex decisions into a sequence of smaller, more manageable components (12, 28). The purpose of the entire process is, of course, to come to a conclusion regarding the best geometric design improvement from a safety standpoint and one satisfactory from other viewpoints such as traffic flow and cost. This decision is usually termed an action in decision theory, and the alternative actions correspond to alternative geometric designs.

To decide on the best action requires information on the value of the alternative actions. Usually in engineering design this information is gathered through the use of predictive and evaluative models, tempered by the engineer's judgment. This gathering of information on the value of alternatives is termed in decision theory an experiment, and the resulting information is called the experimental outcome. In this application, the alternative experiments are alternative predictive and evaluative models, and the outcomes are evaluations of the various alternative designs.

Because the experiments are performed and the outcomes observed before the best action is decided on, the sequence of the steps is as shown in Figure 1. This type of diagram is called a decision tree. The example deals with the problem of improving the safety performance of a short section of rural road with a complex alignment through

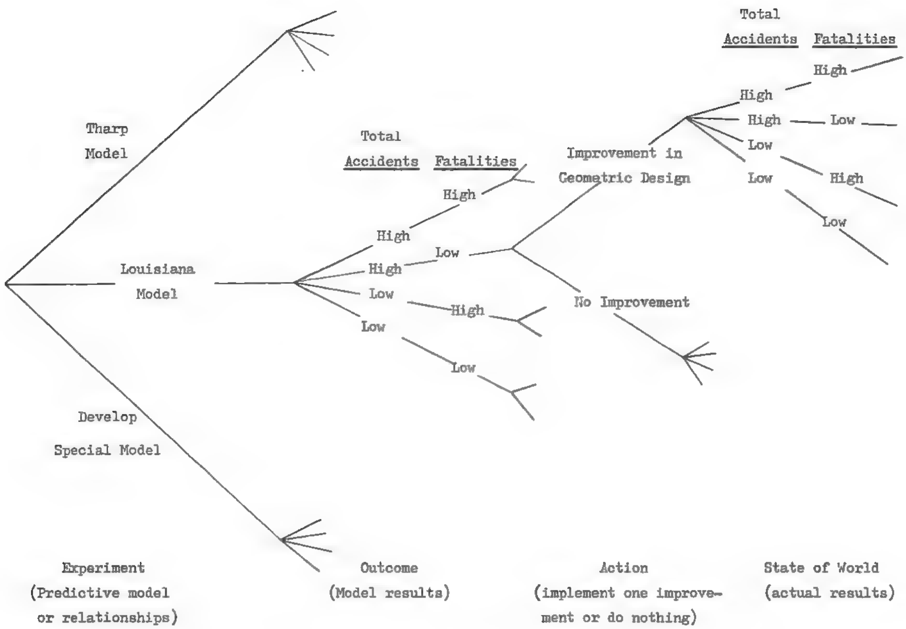

Figure 1. Decision tree for evaluating geometric design improvements.

geometric design changes. For simplicity, this example is limited to 2 alternative actions: (a) improve alignment and profile and install a median barrier and (b) make no improvement. (In an actual study there would be more alternatives.) The decision on action is to be made on the basis of whether there is a significant reduction in total accidents and in fatalities.

The first decision to be made is how to predict the accident rates for each alternative. Three experiments are shown in Figure 1: the Kihlberg and Tharp model, the Mann and Dart (Louisiana) model, or a new model. Each line emanating from the left point represents one experiment. Emanating from each experiment are the 4 possible outcomes if the improvement were made, corresponding to combinations of high or low accidents and fatalities. If no improvement were made, presumably the existing rate would continue (approximately).

After an experiment is performed, the outcome is known, and the action can be selected. Because of space limitations, the alternative actions are shown after only one outcome, but they would all emanate from each.

Only after an action is actually undertaken—such as installing a median barrier—will the true outcome (as opposed to that predicted by the experiment) be known. Because it is desirable to include reference to possible deviations from predictions, the framework includes the possible actual results after actions. These are called states of the world and are shown in this simple example as emanating from only one action; they are possible after each.

This completes the decision theory framework. The first decision to be addressed is that of which experiment to perform. Once the experiment is selected, it is performed and the outcome observed. Then the best action, based on the information available, can be selected and implemented. The means for making these decisions is presented in the following section.

Information Needs

The information needed to employ decision theory is a combination of objective information and subjective information; the engineer draws on his experience in a manner

appropriate to his problem. Although the mathematics of the theory cannot be presented here, it is important to understand the conceptual basis of it.

The first type of information needed is an overall measure of the value of each possible sequence of an experiment, outcome, action, and state of the world; this measure is called a utility. For this problem, utilities include the costs associated with use of and possible development of a model; the costs of implementing an action, either geometric changes or no changes; and the costs and benefits associated with the resulting accident rate, presumably in comparison with the present condition. In the example shown in Figure 1, there would be 24 such utilities.

The other information is in the form of probabilities. One set consists of the probability of each state of the world occurring, given an action prior to the using of any models. For safety problems, these undoubtedly would be based on the judgment of the engineers. The second is information about the predictive accuracy of the various models or experiments. One form is the probability that an experiment will yield a particular outcome, given that a state of the world is true. If there is a high probability that the outcome of an experiment will correspond to the state of the world, then the experiment or model is an accurate one. Again, in this case, such information is likely to be based on the engineer's judgment. This may seem arbitrary, but at least the method takes into account the accuracy of models, and the judgmental factor is made explicit.

Results

This information and the mathematical techniques of decision theory are used in the method first to identify that experiment for which the expected utility is greatest. Then that experiment is performed, and its outcome is observed. At this point the best action is selected and implemented. These are the primary results of use to the engineer.

In addition, other useful information may be obtained. For example, the increase in expected utility resulting from developing better predictive models is easily obtained. Information is provided on a range of possible outcomes resulting from an action and not from just a single "best estimate." However, rather than discuss theory further, we return to actual applications.

EXAMPLE APPLICATION

The example involves a rural site in the Midwest shown in Figure 2. The section is 0.321 mile long, has 4 lanes, is undivided, has no access control, and has reasonably complicated alignment. Its geometric and operating features are given in Table 1. The

TABLE 1

GEOMETRIC AND OPERATING CHARACTERISTICS OF RURAL ROAD SECTION FOR EXAMPLE PROBLEM

Characteristic	Description	Characteristic	Description
Number of lanes, undivided	4	Present speed limit, mph	50
Access control	None	Present ADT	22,000
Length of section, miles	0.321	1980 forecast ADT	29,000
Pavement type	PCCP	Trucks, percent	15
Lane width, ft	11	Present peak-hour volumes	
Shoulder type	Gravel	Westbound	1,700
Shoulder width, ft	8	Eastbound	1,400
Grade, percent	3	Present peak-hour volume-capacity ratio	0.5
Length of grade down in westbound direction, miles	0.26	Present total accident rate/mvm	6.71
Curve radius, ft	1,000	Present fatal accident rate/mvm	0.024
Curve angle, deg	5.7		
Number of traffic conflict points	12		
Number of obstructions	31		

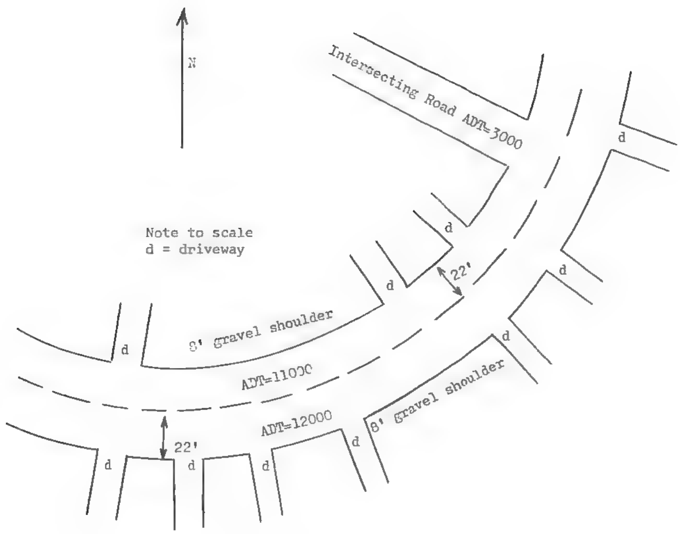

Figure 2. Midwest road section.

problem was formulated in a decision theory framework described in the following.

Experiments

The following possible sources of information on accident rates with 1980 projected traffic and conditions were considered as experiments: experiment 1—use Kihlberg and Tharp log regressions; experiment 2—use Mann and Dart multiple regressions; and experiment 3—develop new predictive model.

Outcomes and States of the World

The possible outcomes and states of the world were arranged to identify thresholds of a total accident rate and a fatality rate that were significantly different from the average rate for a facility type. This average could be the present average or the predicted average for some future year. In this case, the present statewide averages for 4-lane, undivided facilities were used: 7.0 total accidents/mvm and 0.03 fatal accidents/mvm. The thresholds of significantly different rates were identified by using the following quality control formula (13):

$$\lambda_p = \lambda_c + 1.65\sqrt{\frac{\lambda_c}{E}} - \frac{1}{2(E)} \tag{4}$$

where

λ_p = critical rate of significant difference at a 95 percent confidence level;
λ_c = average rate for facility type; and
E = total annual exposure, million vehicle miles (mvm).

Solving this equation with the averages yielded the following: λ_p = 9.3 total accidents/mvm and 0.38 fatal accidents/mvm.

States of the world and outcomes were formed of all possible rate combinations that could possibly influence geometric design. These included the following:

State or Outcome	Total Accident Rate/mvm	Fatal Accident Rate/mvm
1	≥9.30	≥0.38
2	≥9.30	<0.38
3	<9.30	≥0.38
4	<9.30	<0.38

Thus, the set of all relevant possibilities ranges from a significantly high total and significantly high fatal rate, a high total and low fatal rate, a low total and high fatal rate, to a statistically insignificant total and fatal rate.

Alternatives

The information on operating characteristics given in Table 1, investigation of enforcement accident report summaries, and visits to the site revealed that the predominant accident types were rear-end, left-turn, and head-on. All fatals were head-on accidents, and rear-end and left-turn accidents were associated with the presence of driveways and poor skidding resistance on the section. In terms of long-range preliminary planning, the following 4 alternatives were considered:

1. Resurface to appropriate skid resistance standards;
2. Reconstruct section with improved alignment and profile, install partial access control, and eliminate direct driveway access through provision of parallel access facilities to the intersecting road;
3. Make same improvements as in alternative 2 but, in addition, install median barrier and selected median openings; and
4. Make no improvement.

Subjective Information

Based on the traffic projections and past accident history of the site given in the Appendix, a subjective prediction was made of significantly high total rates with equal likelihood of significantly high fatal rates. Also, comparison of the results of all predictors with actual rates at other sites led to the subjective development of probabilities of experimental outcomes for each state of the world that might exist. Specifically, the Kihlberg and Tharp (17) regressions appear reasonably accurate in forecasting so that the outcome of the experiment should correspond closely to the state of the world. In contrast, the Mann and Dart (21) regressions seem to underestimate the rates substantially.

Utilities and Rewards

There are 2 basic components of the utility structure of a model such as this one. One component is that associated with the labor and management cost of the experiment and with the penalty for degree of error in prediction. In our case, error cost can be substantial. Hence there is a utility, U_1, that represents the penalty for error resulting from the use of an experiment in prediction of an outcome that differs from the actual state of the world. The second component utility, U_2, represents the benefits and costs resulting from various actions taken under different states of the world. These are the benefits and costs usually addressed in studies of the value of accident reductions.

Several approaches may be used to arrive at this latter utility component. One is the traditional highway engineering evaluation of net benefits in monetary units resulting from benefits of accident reduction, benefits or costs or both in traffic operations, and capital improvement costs. For each combination of state of the world and alternative action, the resulting annual capital cost, changes in operating and maintenance costs, and reduction in total dollar costs of accidents yield a utility measured in dollars and representing many points of view. An alternative would be to consider each affected group or point of view separately, without necessarily converting all gains and losses into monetary units, and then to assign a utility to each alternative and state of the world combination. Regardless of the approach, a utility measure of value is needed.

For the sake of convenience in demonstrating the model, the total utility of a combination of experiment, outcome, alternative, and state of the world was assumed to be additive and linear on U_1 and U_2; i.e., $U_T = U_1 + U_2$, with U_1 indexed on a scale of 0 to 70 and U_2 on a scale of 0 to 80. However it should be pointed out that either U_1 or U_2 may be a nonlinear complex functional form and that $U_T = f(U_1, U_2)$ may likewise be quite complex. Appropriate inputs for developing U_2 are given in Table 2, and a partial list of utilities is given in the Appendix.

TABLE 2
INPUTS TO EVALUATION OF U_2

Alternative	Annual Capital Cost ($)	Change in Annual Maintenance Cost ($)	Change in Annual User Cost	Change in Accidents if Rate Is Significant (percent)	
				Total	Fatal
1	5,000	-1,000	Negligible	-35	-26
2	20,000	-1,200	Negligible	-40	-30
3	25,000	-500	Negligible	-40	-50
4	0	-1,000	No change	+24	+100

Note: Life = 15 years; interest rate = 6 percent.

Evaluation

The problem was evaluated with SBDT, a computer program developed for this study. Data given in Table 3 show that use of experiment 1, the Kihlberg and Tharp regression, is optimal. The results of experiment 1 are also given in Table 3 and predict a significantly high total and fatal rate. In this particular example it is optimal to take action 2 regardless of the outcome of the experiment, although in the general case the optimal action could depend on the experimental outcome.

Sensitivity Analysis

An important component of any evaluation is the study of the sensitivity of decisions to changing values of pertinent parameters. This is especially important when model inputs are uncertain. One desirable feature of this model and computer program is that it allows one to examine the effects of uncertainty on optimal decisions very easily. If the sensitivity analysis reveals that the same decision (in this case, first the experiment and then the action) is best regardless of the values of the uncertain parameters (within the possible or likely range), then the single best decision can be identified. However, if the optimal decision varies with these uncertain values, then no single decision is best and the range of variation in the uncertain parameters should be reduced through more information and better estimation.

In this example, the sensitivity analysis indicated that the optimal experiment and action described earlier are best over the likely range of uncertain input parameters. The Kihlberg and Tharp experiment is best over all ranges of the probability of each of the states of the world, except when the probability of insignificant accident rates (state 4) is greater than 0.5. Similar results were obtained for the sensitivity analyses of the probabilities describing the prediction accuracy of the models. For all reasonable levels, the Kihlberg and Tharp model appeared best for this particular site. The development of a special model never became optimal, for the costs were too high.

TABLE 3
EVALUATION RESULTS

If Experiment	Yields Outcome	Take Action
1	1	2
1	2	2
1	3	2
1	4	2
2	1	2
2	2	3
2	3	2
2	4	2
3	1	2
3	2	2
3	3	2
3	4	2

Item	Quantity
Number of experiments	3
Number of outcomes	4
Number of actions	4
Number of states	4
Total accidents/mvm	12.19565
Total injury-producing accidents/mvm	4.62885
Total property-damage-only accidents/mvm	7.84199
Total fatalities/mvm	0.05837

Note: Experiment 1 yields optimum 108.050 and outcome 12.20, 0.06.

CONCLUSIONS

In the research underlying this paper an attempt has been made to develop an engineering design methodology that is responsive to the needs of highway engineers concerned with improving traffic safety through geometric design changes. Such a methodology should explicitly consider the fact that accident relationships are not perfect predictors and that many such relationships, each giving different results, may apply to a given problem. Also, the design engineer often possesses much knowledge, based on his experience and judgment, that should be used in the selection of predictive relationships, the evaluation of alternatives, and the final selection of a design alternative. The framework based on decision theory does deal with these aspects of the problem.

The application described here and others have demonstrated the efficacy of this approach. The availability of the associated computer code used in these applications should make further application and use by operating agencies possible. Furthermore, the same framework should be applicable to other highway design and management problems both within and outside of the field of safety.

ACKNOWLEDGMENT

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APPENDIX

INFORMATION FOR EVALUATION

Given information is shown in Figure 3 and computed information in Figures 4, 5, and 6. In all figures, $Q = \theta$.

MARGINAL MEASURES ON Q

Q1	.300
Q2	.300
Q3	.200
Q4	.200

CONDITIONAL MEASURES ON Z EXPERIMENT

	E1				E2				E3			
	Q1	Q2	Q3	Q4	Q1	Q2	Q3	Q4	Q1	Q2	Q3	Q4
Z1	.30	.40	.20	.25	.25	.15	.20	.10	.40	.30	.25	.25
Z2	.30	.30	.20	.20	.25	.25	.20	.10	.20	.30	.25	.25
Z3	.20	.20	.50	.25	.25	.25	.20	.30	.20	.20	.25	.25
Z4	.20	.20	.10	.25	.25	.35	.40	.50	.20	.20	.25	.25

Figure 3. Given marginal and conditional measures.

MEASURES ASSOCIATED WITH E1

Z	Joint Q1	Joint Q2	Joint Q3	Joint Q4	Marg on Z
Z1	.090	.120	.040	.050	.300
Z2	.090	.090	.040	.040	.260
Z3	.060	.060	.100	.050	.270
Z4	.060	.060	.020	.050	.190

MEASURES ASSOCIATED WITH E2

Z	Joint Q1	Joint Q2	Joint Q3	Joint Q4	Marg on Z
Z1	.075	.045	.040	.020	.180
Z2	.075	.075	.040	.020	.210
Z3	.075	.075	.040	.060	.250
Z4	.075	.105	.080	.100	.360

MEASURES ASSOCIATED WITH E3

Z	Joint Q1	Joint Q2	Joint Q3	Joint Q4	Marg on Z
Z1	.120	.090	.050	.050	.310
Z2	.060	.090	.050	.050	.250
Z3	.060	.060	.050	.050	.220
Z4	.060	.060	.050	.050	.220

Figure 4. Computed measures.

REVISED CONDITIONALS FOR Q WITH E1

Z	Q1	Q2	Q3	Q4	SUM
Z1	.300	.400	.133	.167	1.000
Z2	.346	.346	.154	.154	1.000
Z3	.222	.222	.370	.185	1.000
Z4	.316	.316	.105	.263	1.000

REVISED CONDITIONALS FOR Q WITH E2

Z	Q1	Q2	Q3	Q4	SUM
Z1	.417	.250	.222	.111	1.000
Z2	.357	.357	.190	.095	1.000
Z3	.300	.300	.160	.240	1.000
Z4	.208	.292	.222	.278	1.000

REVISED CONDITIONALS FOR Q WITH E3

Z	Q1	Q2	Q3	Q4	SUM
Z1	.387	.290	.161	.161	1.000
Z2	.240	.360	.200	.200	1.000
Z3	.273	.273	.227	.227	1.000
Z4	.273	.273	.227	.227	1.000

Figure 5. Revised conditionals.

PARTIAL LIST OF UTILITIES

E	Z	A	Q	U_T	U_1	U_2
1	1	1	1	85	65	20
1	1	1	2	90	50	40
1	1	1	3	105	40	65
1	1	1	4	60	30	30
1	1	2	1	120	65	55
1	1	2	2	125	50	75
1	1	2	3	110	40	70
1	1	2	4	50	30	20
1	1	3	1	145	65	80
1	1	3	2	100	50	50
1	1	3	3	115	40	75
1	1	3	4	40	30	10
1	1	4	1	70	65	5
1	1	4	2	55	50	5
1	1	4	3	45	40	5
1	1	4	4	105	30	75
1	2	1	1	80	60	20
1	2	1	2	105	65	40
1	2	1	3	95	30	65

E	Z	A	Q	U_T	U_1	U_2
1	2	1	4	60	30	30
1	2	2	1	115	60	55
1	2	2	2	140	65	75
1	2	2	3	100	30	70
1	2	2	4	50	30	20
1	2	3	1	140	60	80
1	2	3	2	115	65	50
1	2	3	3	105	30	75
1	2	3	4	40	30	10
1	2	4	1	65	60	5
1	2	4	2	70	65	5
1	2	4	3	35	30	5
1	2	4	4	105	30	75
1	3	1	1	70	50	20
1	3	1	2	70	30	40
1	3	1	3	130	65	65
1	3	1	4	75	45	30
1	3	2	1	105	50	55
1	3	2	2	105	30	75
1	3	2	3	135	65	70

Figure 6. Utilities.

ACCESS AND PARKING CRITERIA FOR HOSPITALS

V. Setty Pendakur and Paul O. Roer, University of British Columbia

Eleven public general hospitals in metropolitan Vancouver, having a total capacity of 2,700 beds and providing direct employment for 5,150 people, were included in this study. The main focus of the study was to describe and understand employee traffic and parking needs with a view to developing overall planning standards for access and parking. The study showed that most employees live within 2 miles of their hospitals and that nurses and technical employees tend to live within 1.6 miles. The employees of larger hospitals live closer to their hospitals than those at smaller hospitals. On the average, 58 percent come to work by car, 20 percent by bus, and 22 percent on foot. Parking is inadequate at most hospitals, and hospital users and employees park on adjacent streets. Trip generation characteristics and travel mode at each hospital were studied in the framework of hospital size, location, and level of transit service. Based on socioeconomic characteristics of employees, parking needs of this group were developed in relation to hospital size and transit service. Access requirements of hospitals are discussed. Finally, visitor and patient requirements deduced from other studies are used to develop parking standards for future planning.

•BY 1980, approximately 20 million Canadians, equal to 80 percent of the total population, are expected to live in urban areas, compared with 73 percent in 1967. The distribution of this urban population is further expected to resemble an east-west demographic ribbon parallel to the U. S. border. Of particular importance is the forecast that most of the predicted urban growth will take place only in a few cities, especially Montreal, Toronto, and Vancouver (1).

Hospital and health care facilities will grow not only because the population is increasing but also because a greater variety of health services is being offered and is available to a greater proportion of the population through a national commitment to universal health care. These facilities and their use are expected to grow faster than population itself.

Recent population forecasts for metropolitan Vancouver indicate a growth pattern of 892,000 people in 1966 (estimated at 915,000 in 1968) increasing to 1,169,000 in 1976 (2). On the other hand, the number of hospital beds is expected to increase by almost 50 percent during 1968 to 1976 even though the number of hospital employees is not expected to increase at a higher rate than the total population in metropolitan Vancouver.

In 1960, Smith examined several hospitals in 8 major U. S. cities and recommended planning standards for parking (3). He examined several traffic and parking studies conducted in major U. S. cities together with related zoning and parking standards. The basic unit of measurement used by Smith was the number of beds in the hospital, and the focus was on parking requirements. Smith's study has provided very useful guidelines for planning of urban and suburban hospitals. There is, however, a critical and urgent need to update these standards in view of the changing nature of hospital services and their changing trip generation characteristics.

In 1969, Keefer and Witheford studied 78 hospitals in 16 metropolitan areas in the United States with particular reference to their trip generation characteristics (4).

Sponsored by Committee on Parking and Terminals.

Travel data examined by Keefer and Witheford were derived from a number of traffic and transportation studies done on a metropolitan area-wide basis. The study examined overall employee trip characteristics, trip lengths, trip mode, and peak-hour characteristics related to employees and visitors. These data need to be supplemented by an examination of the trip generation characteristics of hospitals and their linkages to socioeconomic characteristics of employees and their housing location in the Canadian context.

The Vancouver metropolitan area with a population of almost 1 million people has 15 public hospitals fully supported by local, provincial, and federal governments. They have a total capacity of 4,400 beds, employ 9,000 people, and are estimated to provide support for a population of 100,000 including those employed by hospital-related industry. By 1976, these hospitals expect to employ 14,000 people directly with a total estimated bed capacity of 6,200. The research presented here examines the journey-to-work patterns of employees and their relationship to the spatial distribution of housing within the urban structure vis-à-vis socioeconomic characteristics of employees and available transportation linkages (automobile and transit).

STUDY SCOPE AND METHODOLOGY

Study Hospitals

Only 11 general hospitals in metropolitan Vancouver were included in this study. The following 3 hospitals were excluded because of their uniqueness: Children's Hospital, G. F. Strong Rehabilitation Center, and Vancouver General Hospital. Children's Hospital and the G. F. Strong Center are special hospitals catering to an identifiable and unique group of patients and quite unlike other general-purpose hospitals. The Vancouver General Hospital is the biggest hospital in the province of British Columbia and caters to the special and general needs of the entire province. Peace Arch District Hospital was new and not yet in full operation at the time of data collection.

Hospital Characteristics

Characteristics of hospitals included in the study are given in Table 1. Spatial location, interrelationships, and size are shown in Figure 1. These hospitals range in capacity from 52 to 551 beds and in employment from 65 to 1,142 persons. All are public hospitals and derive support from local, provincial, and federal governments.

Only one hospital—Surrey Memorial—is not served by transit. For the remaining ten, the transit service varies from 1 bus line to 16 bus lines within 1,000 ft of walking distance. St. Paul's Hospital, one of the largest, is located in the central business district of Vancouver. As can be expected, it has excellent transit service. Mount St. Joseph's Hospital, outside the CBD but in the heart of an old business district, also has excellent transit service. All other hospitals are served by transit and have available 1 to 6 bus lines within 1,000 ft of walking distance.

Parking provided at these hospitals varies from 35 to 445 spaces. Only one hospital provides parking as required by the zoning bylaw, three provide more than that required, and seven provide less than that required.

Scope of Study

The study deals with employee travel and related facilities only; it does not, therefore, present a total picture of hospital transportation characteristics and needs. A complete picture must include consideration of the needs and characteristics of patient, visitor, and delivery transport in addition to those of employees.

The study is descriptive and does not pretend to be deterministic for cause and effect relationships. The findings should prove useful, however, to hospital administrators and planning agencies concerned with the planning of new or expanded hospital facilities or with the upgrading of present transportation facilities.

TABLE 1

HOSPITAL CHARACTERISTICS

Hospital Capacity and Employment							Cost of Parking Development		Parking Spaces Provided			Parking Required by Local Zoning Bylaw	
Name	Type[a]	Number of Beds 1968	Number of Beds 1976	Gross Floor Area (ft^2)	Number of Employees	Transit Service Index[b]	Land	Improvements	Staff	Public	Total	Space Ratio	Number of Spaces
Grace	CG	93	170	67,807	235	L	64,000	—	15	36	51	1/1,000 gross ft^2	68
Holy Family	Reh	52	52	24,500	65	M	2,500	2,500	10	25	35	1/1,000 gross ft^2	25
Lions Gate	CG	484	658	263,000	707	M	—	—	73	0	73	1/4 beds	121
St. Joseph	CG	141	400	72,000	166	H	82,000	5,000	—	—	71	1/1,000 gross ft^2	72
Richmond General	CG	154	229	101,420	199	M	12,400	—	26	114	140	1/1 bed	154
St. Mary	CG	256	256	157,848	321	M	26,018+	11,296+	67	30	97	1/1,000 gross ft^2	158
St. Vincent	CG	180	332	95,000	297	M	50,000	10,000	35	60	95	1/1,000 gross ft^2 1/4 beds	95
Surrey Memorial	CG	103	253	47,600	164	L	—	—	107	28	135	1/1 staff doctor 1/3 employees	81
Royal Columbian	Ref	434	403	325,637	869	L	200,000	15,000	279	166	145	1/1,000 gross ft^2	326
St. Paul	Ref	551	651	247,500	1,142	H	—	—	161	4	165	1/1,000 gross ft^2 1/5 beds	247
Burnaby General	CG	242	350	168,778	379	L	46,000	18,000	70	155	225	1/2 staff doctors 1/4 employees	147

Note: 1968 data unless otherwise stated.

[a]CG = community general, active care; Reh = rehabilitation; and Ref = referral, approximately 30 percent of cases from outside city and approximately 20 percent of cases from outside metropolitan area.

[b]L = 0, 1, or 2 bus lines within 1/5 mile; M = 3, 4, 5, or 6 bus lines within 1/5 mile; and H = 7 or more bus lines within 1/5 mile.

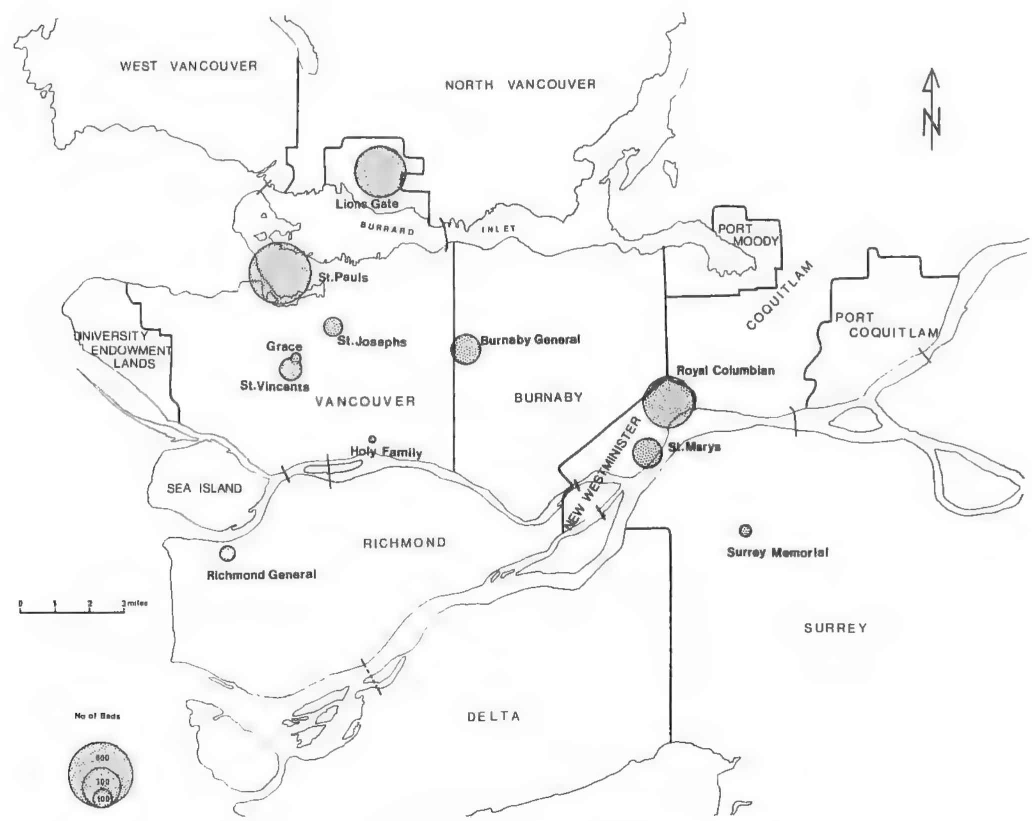

Figure 1. Location and size of hospitals in study.

Study Methodology

Information on employee travel and other characteristics was obtained in August-September 1968. Questionnaires were distributed to 5,150 employees through the payroll offices in the 11 hospitals. Forty-eight percent or 5,200 employees returned the completed questionnaires; 3 hospitals returned 100 percent samples; and 1 hospital returned only a 20 percent sample.

Information on basic hospital characteristics was obtained in November-December 1968. This information was compiled from the files of the respective hospitals, the Greater Vancouver Regional Hospital District, and the local municipal planning departments. Only simple statistical and graphical analysis techniques have been used to discover apparent correlations between the variables considered.

TRAVEL CHARACTERISTICS

Parking has become a major issue facing the administrator of an urban hospital; with both land and capital in short supply, he is often unable to provide the amount of parking that seems to be required. Municipal bylaw requirements for hospital parking are often inadequate and add to the administrator's problems because more parking than that legally required is not likely to be constructed until severe problems develop.

Parking demand determinants at urban hospitals are directly related to the type and magnitude of health care provided. Whether it be a general, referral, or special care hospital, the number of beds determines the number and type of employees as well as visitor potential and emergency needs. To some degree, aggregate parking demand is a direct function of the number of beds or the number of employees or both.

Trip Generation

Each employee generates at least 2 trips per day, 1 trip to and 1 trip from the hospital. Exceptions to this are the doctors who may travel to and from a hospital several times a day for consultation and emergencies. Employee trip-making, other than to and from work, was not considered in this study, but the assumption of 2 trips per employee, with some additional allowance for doctors, was considered adequate for planning purposes. Visitor trips, which account for most of hospital travel, must also be considered.

Shift Work

Shift work reduces the proportion of employees present at any one time; the total number of hospital employees, therefore, may not be the best indicator of the demand for parking. The study data indicate that, on the average, 50 percent of employees do not work shifts at all and that 79 percent of employees work during the daytime. The proportion of employees working each shift is as follows:

Shift	Percentage of Employees
None	50
Day	29
Evening	14
Night	7

The most critical time of day for employee parking is the period immediately before the start of the evening shift. During this period parking space must be available not only for those working the evening shift but also for those working the day shift and straight days. An average of 14 percent more parking space than would normally be required by employees must be available to handle the critical period unless an effective system of staggered shift changes is in operation.

Parking Cost

Only 1 of the 11 hospitals included in this study charges for the use of its parking space. These charges are very moderate, in the order of 10 to 15 cents per hour or

$6 per month, and certainly insufficient to reduce the demand for parking to a degree that can be observed in parking-lot usage. The capital cost of providing off-street parking varies with the location of the hospital, from a low of $150 to a high of $1,250 per space. The average cost per space is $500. This is between 2 percent and 2.5 percent of the capital cost of hospital construction with a "1-bed, 1-space" policy.

Residence Location

Approximately 50 percent of all hospital employees live within 1.8 miles of their places of work. This distance is different for each hospital. Employees of large hospitals tend to live closer to work than those at small hospitals. This is shown in Figure 2. (All distances are measured in straight air-line miles.)

The median home-work distance for hospital employees driving cars to work was found to be 2.8 miles. This compares favorably with the patterns of journey to work in metropolitan Vancouver, although hospital employees seem to live closer to work than others. Wolforth's study of journey to work in Vancouver indicated a median home-to-work distance of 3.4 miles for work centers outside the CBD, while Pendakur and Hickman pointed out that mean journey-to-work distance for peripheral work centers is 3.6 miles. Wolforth and Pendakur measured over-the-road distances, and these are generally higher than air-line distance used in this study (5, 6).

Hospital employee work-trip lengths for automobile drivers, as revealed in this study, are slightly less in Vancouver than in the United States. Keefer and Witheford, in a study of 78 hospitals that included hospitals of 150 to 600 bed capacity, showed that average work-trip length was 3.5 air-line miles (4). Work-trip lengths are generally lower in Vancouver than in other large U.S. cities because it is a smaller urban area. This has been studied and pointed out by Wolforth and Pendakur and by Hickman.

The median home-to-work distance for transit riders is 2.2 miles in Vancouver. No comparative data for other work centers in Vancouver for transit riders are available.

Employee category and distance to work are related as shown by data given in Table 2. Because only 4 hospitals returned information concerning doctors, the median distance shown for this group may be biased. If doctors are excluded, there seems to be a correlation between socioeconomic status, as defined by employee category, and the home-to-work distance.

Employees in the nontechnical category appear to have a significantly longer trip to work than do those in other categories. Two arguments may be advanced in explanation: (a) Employees in the lower socioeconomic class cannot afford to obtain housing close to their places of work and must substitute travel in place of convenient locations; or (b) nontechnical work is not unique or peculiar to hospitals, and employees in this category can work in many places and will therefore tend to optimize their locations in terms of all work opportunities. The latter argument is supported by the fact that employees tend to live closer to large hospitals near which there are many other work opportunities.

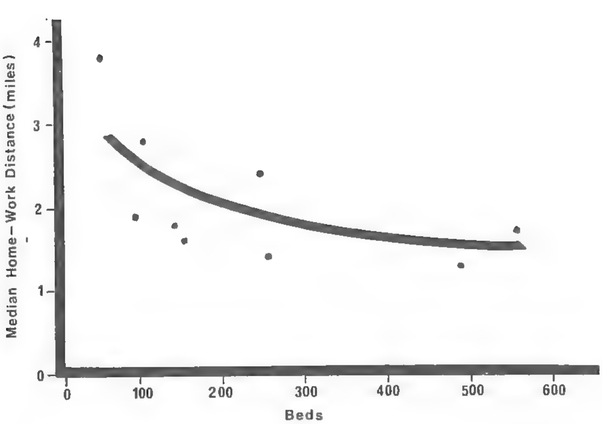

Figure 2. Distance from home to work versus size of hospital based on number of beds.

Modal Split

Modal split varies not only among hospitals but also among employment categories (Table 2). The proportion of employees driving to work varies with socioeconomic status as defined by employment category. All the doctors are automobile drivers. This is

TABLE 2

EMPLOYEE HOME-TO-WORK TRAVEL DISTANCE AND MODE

Employment Category	Distance (miles)	Mode (percentage of employees)			
		Automobile Driver	Automobile Passenger	Bus	Walk and Other
Doctors	2.25	100	0	0	0
Technical	1.61	61	7	15	17
Nurses	1.85	50	10	15	25
Administrative and clerical	1.85	46	11	21	22
Nontechnical	2.11	36	10	31	23
Average		48	10	20	22
Range of average					
High		85	12	33	34
Low		24	5	0	3
Urban hospitals only		46	11	19	24
Suburban hospitals only		68	10	10	12

understandable because of the nature of their work and is consistent with experience in the United States. A very high proportion of technical employees (61 percent) and nurses (50 percent) drive their cars to work compared with administrative (46 percent) and nontechnical employees (36 percent). Those of higher status and in higher income groups drive to work. Keefer and Witheford have shown that 42 to 62 percent of employees drive to work at U.S. hospitals (4). In comparison, 48 percent of all employees drive to work in Vancouver.

Twenty-two percent of all employees live close enough to be able to walk to work. Again lower socioeconomic groups (administrative and nontechnical) tend to live closer to work. Only 17 percent of technical employees live within walking distance to work compared with 23 percent for nontechnical and 22 percent for administrative employees.

On the other hand, 25 percent of the nurses live within walking distance. This may be due to several reasons: Some hospitals have nurses' residences located very close by; most nurses are single and tend to aggregate and live close by; many nurses are required to work shifts; and either apartment or multifamily rental accommodations are available near most hospitals.

The number of employees coming as automobile passengers is fairly constant at an average of 10 percent of all employees. Car occupancy is 1.21 for hospital employees, and this is slightly lower than overall car occupancy computed by Lea (7) of 1.34 for work trips for Vancouver.

Hospital employee travel mode and trip length characteristics presented here seem to be typical and comparable to those in large cities of western Canada. For example, hospital employees in Edmonton, Alberta, have similar trip lengths and use similar modes if allowance is made for the colder climate that discourages walking. In Edmonton 52 percent of employees drove cars compared with 48 percent in Vancouver, and 16 percent came by bus compared with 20 percent in Vancouver (8).

There are variations in the modal split among hospitals, but 2 distinct patterns can be identified. One pattern is best described as urban; 4 hospitals fit this pattern. The other pattern is best described as suburban; 3 hospitals fit this pattern. Modal-split variation between urban and suburban hospitals is given in Table 2. Hospitals described as urban are typically located in fairly intensely developed areas (6,000 persons/sq mi or more) and have average or better than average transit service. Hospitals fitting into the suburban pattern are typically located in less intensely developed areas (less than 6,000 persons/sq mi) and have less than average transit service.

Parking

The amount of parking required by employees varies with staff composition and shift-work practices and is related to alternative transportation quality available. On the

average, all doctors, 61 percent of technical employees, 50 percent of nurses, 46 percent of administrative and clerical employees, and 36 percent of nontechnical employees drive to work and require parking. Overall, it was found that 48 percent of employees require parking, but this proportion varies between 24 and 85 percent (Table 2). Not unexpectedly, the low proportion of drivers applies to a downtown hospital with very good transit service and the high proportion to a suburban hospital with almost no transit service.

Approximately 75 percent of employee cars are parked on hospital property, 21 percent on the streets in the neighborhood, and 4 percent on private parking lots. The number of employee cars parked on the neighborhood streets can be reduced considerably by a moderate increase in the amount of off-street parking provided. Forty percent of the hospitals appear to have recognized this and are providing an average of 45 percent more parking than required under the zoning regulations. At the same time another 40 percent of the hospitals are supplying an average of 30 percent less parking than is required. The effect of these variations on on-street parking is as follows:

Amount of Parking Provided as Percentage of Zoning Requirement	Employee Cars Parked in Hospital-Provided Lots (percent)
145, 4 hospitals	95
100, 2 hospitals	82
70, 4 hospitals	64
30, 1 hospital	34

Municipal zoning regulations for off-street parking to be provided by hospitals vary between "one parking space per bed" and "no parking required." Between these extremes another 5 varieties of regulations establish minimum requirements. The regulations for the 14 municipalities in metropolitan Vancouver are given in Table 3.

The wide variety of minimum standards for parking suggests little agreement on the magnitude of the parking problem facing hospitals. For the urban hospitals, an average of 0.57 parking space per bed was required by zoning bylaw; and for the suburban hospitals, an average of 0.78 parking space per bed was required.

The level of transit service to a hospital can be used as an indicator of employee demand for parking. Grouping the same hospitals by the number of bus lines serving them revealed the following:

Transit Service		
Level	Number of Bus Lines Within 1,000 Ft	Employees Driving to Work (percent)
High	7 or more	27
Medium	3, 4, 5, and 6	54
Low	2 or fewer	67

Transit service is generally better in high-density areas, and the effect of transit service suggested by the preceding data also includes the effects of density and income on car ownership and consequently on modal split. Figure 3 shows the relationship among density, income, and car ownership in greater Vancouver. Data shown in Figure 3 are consistent with the trends in other Canadian metropolitan areas. Generally speaking, high-income groups living in low-density areas have the greatest number of cars per household. Almost 100 percent of the hospital employees have annual incomes of more than $3,000. All doctors and some technical employees have incomes of more than $8,000 per year. More than two-thirds of the total employees (technical, nontechnical, administrative, clerical, and nurses) have annual incomes of $5,000 to $8,000. Median car ownership is approximately 1.0 car per household and slightly higher in low-density suburban areas.

Figure 4 shows the relationship between the proportion of employees driving to work and resident population density in the neighborhood where the sample hospital is located. On the average 50 percent of employees live within 1.8 miles of

their hospitals, 67 percent live within 2.7 miles, and 85 percent live within 4.8 miles. Even though these commuting distances vary, it is significant to note that 50 percent of employees live in the hospital neighborhood itself, and consequently their socioeconomic-travel characteristics are typical of those in neighborhoods having similar density, income, and other neighborhood characteristics.

Data given in the preceding tabulation and shown in Figures 3 and 4 are interrelated and should be considered together. For example, hospitals with low transit service are characteristically in the suburbs where there are low densities and slightly higher incomes and where the highest proportion of employees drive to work. In contrast, hospitals with the highest level of transit service are characteristically located either in the central business district or en route to the CBD but not necessarily in low-income neighborhoods. These have the fewest employees who drive and the most employees who ride transit. All the hospitals studied provide similar services, but their locations influence transportation service levels and employee travel habits. The number of employees driving to work is obviously a function of income, car availability, and transit service and directly determines the employee parking needs.

TABLE 3

ZONING REQUIREMENTS FOR HOSPITAL PARKING

Zoning Requirement	Number of Municipalities
1 space/1,000 sq ft of floor area	2
1 space/4 beds	1
1 space/2 beds	1
1 space/1 bed	1
1 space/4 beds, 1 space/1 staff doctor, and 1 space/3 employees	2
1 space/5 beds, 1 space/2 staff doctors, and 1 space/4 employees	1
No requirement	6

Based on the amount of parking actually supplied by the various hospitals, in terms of parking spaces per bed or parking spaces per 1,000 sq ft of floor area, approximately 95 percent of employee cars will be parked in hospital parking lots if these lots have 0.75 parking space per bed or 1.3 parking spaces per 1,000 sq ft of floor area. Provision of parking simply to prevent employee cars from being parked on the street is, of course, no guarantee that the employee demand for parking is being met. It seems reasonable to assume, however, that had a large unsatisfied demand for employee parking existed, the street spaces freed through provision of more off-street parking would automatically be used to satisfy this demand. This has not occurred, and it may be assumed that the employee demand for parking is reasonably satisfied if 0.75 space per bed is supplied. To what extent this amount of parking will satisfy the visitor de-

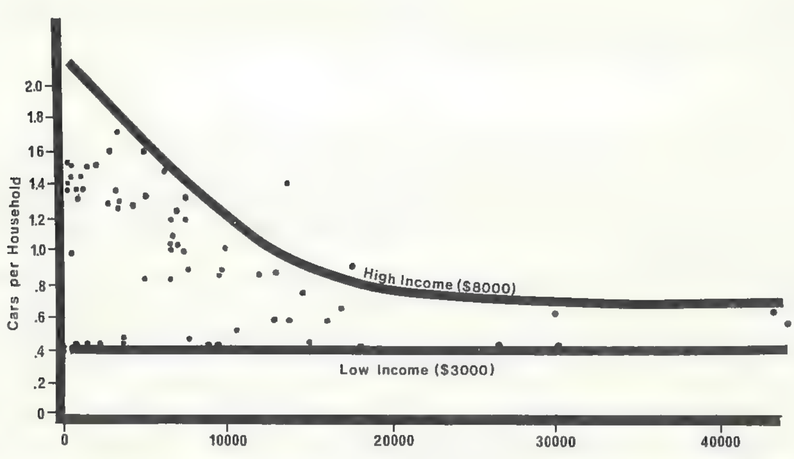

Figure 3. Relationship of population density, income, and automobile ownership.

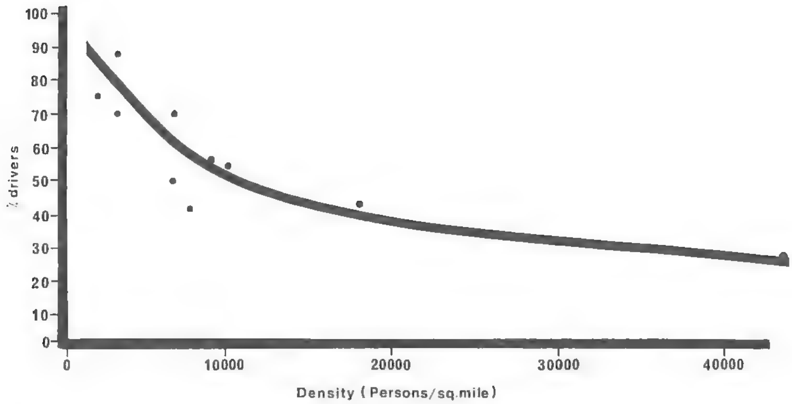

Figure 4. Relation of resident population density in hospital area and proportion of employees driving to work.

mand is uncertain; employees are usually in a position to preempt parking spaces, and the relative satisfaction of their needs is not necessarily a measure of the satisfaction of other users' needs.

PLANNING CRITERIA FOR ACCESS AND PARKING

In 1960, Smith studied 8 large hospitals in the United States and concluded that the hospitals had serious access problems and did not provide adequate parking facilities (3). He recommended interim parking standards and suggested further research. The American Hospital Association subsequently made an effort to incorporate access and planning criteria into hospital development procedures. These standards were minimal and somewhat inadequate even in 1962 (9, 10). Biciunas studied the trip generation potential of hospitals in 1965 in the context of increasing availability and use of health care services and pointed out the need to coordinate transport planning with site planning (11).

In 1969, Keefer and Witheford analyzed travel patterns at 78 large hospitals in the United States as a part of their study of urban travel patterns to major trip generators. They found that on the average parking facilities at hospitals were inadequate and that some hospitals had serious access problems (4).

Changing Trends

Access and parking problems at many Canadian hospitals partly result from tremendous post-war growth and partly reflect the general increase in congestion on urban arterials in Canada during the past 25 years. Because of a national commitment to universal medicare in Canada, there is likely to be unprecedented expansion of health care facilities in the future.

Hospital use has approximately doubled in Canada since 1945. However, the number of new hospitals constructed has only been small. Even though the hospitals have grown larger in bed-care capacity, the average length of stay per patient has decreased. Therefore, even where beds and other new facilities have not been added, patient and visitor trips to hospitals have increased. There are more people working in hospitals now than in the past. Even with mechanization and automation, every medicare advance brings with it the need for more doctors, nurses, and technicians. This has increased the work trips to hospitals.

There has been a continuing shift from transit to the automobile consistent with general urban travel characteristics of an increasingly affluent population. Even when

adequate public transit facilities are available, it is likely that patients and visitors will continue to travel by car. It is likely that public transit may not meet hospital travel requirements, particularly those of the aging population that has increased frequency of trips for medical purposes (12).

Roth, in a study of hospital locations and future needs, concluded that future hospitals will be fewer, but larger, long-term care institutions will be integrated with community hospitals, and the hospital will be the focal point of all health activities in the community (13). With increasing national commitment to health care and increasing public use by a more affluent society, the urban hospitals will become more important as major trip generators and focal points of urban travel.

Criteria for Access

Planning for access to major hospitals, or for that matter to any other major trip generator, cannot be undertaken outside the context of urban transportation planning within the entire metropolitan area. Where the major traffic generators such as hospitals are public facilities, there should at least be fewer problems of communication between hospital management and urban transportation planning agencies. Yet even this study of only 11 hospitals suggests lack of coordination and understanding of access and travel problems and zoning standards on the one hand and implementation of these standards on the other.

There is no question about the basic objective that all hospitals should have adequate access for patients, visitors, and employees. Access includes roads as well as transit. Nevertheless the findings of this study show that one fairly large suburban hospital built recently has no bus lines within reasonable walking distance (½ mile), and no attempt has been made either by the hospital or by the transit agency (B.C. Hydro and Power Authority) to improve the service. This may indicate that both agencies do not consider that transit service is necessary. It must be clearly understood that this implicit level of service gives very little consideration to those who do not have cars.

The principal application of the findings of this study regarding access to hospitals is informational. The objective of the study was to explore the relationship between travel to hospitals and various factors that influence such travel. Furthermore the study was limited to employee travel only. The data presented in this study suggest that transportation planners should concentrate more on major trip generators. Hospital travel can have critical impact on local travel facilities. In the context of increasing use and size of urban hospitals, locational criteria and zoning standards could be seriously reviewed.

Parking Standards

The data presented here depict the current conditions with regard to access and parking at 11 hospitals included in this research. At most of these hospitals, the current supply of parking falls critically short of meeting the current employee parking demand. Often employees park on the hospital lot, preempting visitors and patients. Consequently, employee and visitor cars are often parked on surrounding streets.

The shortage of parking space at urban hospitals appears to be a chronic problem that the hospitals alone cannot solve. At hospitals located in high-density commercial (CBD) or residential areas, convenient parking would help relieve the natural anxieties of hospital trip-making. However, at hospitals located in low-density off-center areas, whether all the required parking should be on the hospital lot and not on the surrounding streets is an issue that each community must resolve. Additional costs relative to the provision of all the required parking by the hospital must be balanced against community values related to street parking and congestion, especially when hospitals are publicly funded.

This study has included an analysis of the employee travel characteristics. To develop proper standards for planning purposes requires further analysis of visitor and patient parking demands. Even though this research has been neither extensive nor conclusive, it is possible to develop parking standards based on current travel patterns of employees and on assumptions of proportional demands for visitors based on experience elsewhere.

TABLE 4

EMPLOYEE PARKING DEMAND AT HOSPITALS

Hospital	Transit Service Level[a]	Number of Employees	Maximum Daytime Total[b]			Employee Cars During Day Shift per 100 Employees	
			Employees	Automobile Drivers (percent)	Automobiles	Number	Average
St. Joseph	H	166	131	40	52	32	25
St. Paul	H	1,142	904	24	216	19	
Holy Family	M	65	54	39	21	33	42
Lions Gate	M	707	523	58	303	43	
Richmond General	M	199	144	72	104	52	
St. Mary	M	321	254	47	119	37	
St. Vincent	M	297	255	53	135	45	
Grace	L	235	162	51	83	35	51
Royal Columbian	L	869	756	67	506	58	
Surrey Memorial	L	114	144	85	97	59	

[a]H = 7 or more bus lines within 1/5 mile; M = 3 to 6 bus lines within 1/5 mile; and L = 0 to 2 bus lines within 1/5 mile.
[b]Includes all employees working day shift and those working no shifts but regularly during days.

Visitor parking space requirements are directly related to the number of patients served by the hospital. Keefer and Witheford, in analyzing the parking provided at 29 general hospitals of up to 500 bed capacity, found that on the average 22 parking spaces per 100 beds were provided for visitors (4). Based on the experience by Smith and Biciunas, this seems adequate but may be low in view of predictions by Ballard and Roth (3, 11, 12, 13). The 11 hospitals included in this study were all general hospitals and on the average had 1.5 to 2.0 employees per bed. Based on this, the U.S. experience shows that the patient and visitor parking needs are approximately 10 to 15 parking spaces per 100 employees. Because of current shortages of parking spaces, the changing trends, and the possibly increasing use of automobiles for patient and visitor travel discussed earlier, it is assumed for the purpose of developing parking standards that the patient and visitor parking needs are 10 to 20 spaces per 100 employees depending on the quality of alternative transportation service available.

In the assessment of the employee parking demand, it is assumed that the objective is to provide parking for all employees who are likely to drive their cars as determined by current socioeconomic conditions. The study has not included any consideration of increased parking fees that may restrain parking demand. Maximum employee accumulation is assumed to be during daytime after the arrival of clerical and administrative employees who generally do not work shifts and the employees who work the day shift. No allowance has been made for shift overlaps as it is presumed that there is sufficient staggering of arrivals and departures to take care of overlap.

The availability, quality, and quantity of transit service within walking distance clearly affect employee parking demand. Data given in Table 4 show that employee parking demand varies between 25 and 51 spaces per 100 employees depending on transit service quality.

Recommended parking standards are given in Table 5. Although these general standards may be applicable to general hospitals of 50 to 500 bed capacity, they must be used only as guidelines in planning. The requirements for each hospital must be developed within the context of urban area transport service levels, community values, and unique characteristics of each hospital. The standards given in Table 5 are a result of analysis of employee travel

TABLE 5

PARKING STANDARDS FOR GENERAL HOSPITALS OF 50 TO 500 BEDS

Transit Service Level[a]	Parking Spaces Required per 100 Employees		
	For Employees	For Visitors	Total
High[b]	25 to 30	10	35 to 40
Medium	40 to 45	10 to 15	50 to 60
Low[c]	50 to 60	15 to 20	65 to 80

Note: Full-time employee equivalent: 1 bed = 1.5 to 2.0 employees.
[a]High = 7 or more bus lines within 1/5 mile; Medium = 3 to 6 bus lines within 1/5 mile; and Low = 0 to 2 bus lines within 1/5 mile.
[b]Applicable to hospitals within or near either the CBD or very high density areas.
[c]Applicable to suburban hospitals with little or no transit service in very low density areas.

characteristics to 11 general hospitals in the Vancouver metropolitan area with a total population of approximately 1 million people. Included in the study were 5,200 employees, and the analysis involved simple statistical correlations. Finally, when parking requirements of hospitals are developed, public policy determinants and price-demand functions must also be considered before investment decisions are made regarding planning for parking. Although this study included 11 hospitals and the related access and parking criteria, it has by no means been exhaustive. However, the parking standards given in Table 5 are applicable to general hospitals in Canadian metropolitan areas; modifications can be made for each unique case.

FUTURE RESEARCH

Future research should include components of parking policy formulation, visitor and patient parking requirements, attitudes to public transport, and transport systems interpretations. Access and parking policies of the hospital must be developed in the context of urban transportation planning. Long-range national land use plans in the hospital neighborhood may reduce vehicular travel of employees.

Because major hospitals occasion as many trips as some central business districts and create circulation problems, the planning of related traffic facilities must be done on a cooperative basis between hospitals and planning agencies. This appears, however, not always to be the case. To resolve some of the resulting problems will require guidelines for planning not only locally but also regionally and nationally. Research aimed at disclosing variations in existing planning policies should make an interesting and effective beginning.

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ANALYTICAL AID FOR EVALUATING HIGHWAY AND ROADSIDE GEOMETRICS

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A mathematical model is described that represents the dynamic motions of a motor vehicle in contact with a variety of roadway and roadside geometric design features including fixed obstacles. The model has 15 degrees of freedom, and all major nonlinearities are included in the motion equations that have been programmed for time-history solutions on a digital computer. The task of interpreting the extensive output information from the model is eased by a computer-graphics display technique that produces detailed perspective drawings of the vehicle and terrain at selected intervals of time during a simulated maneuver. Sample comparisons are presented of analytically predicted vehicle responses and full-scale test results. A description of the validation approach followed in this research effort is included. A series of sample applications is described, and the capabilities of the model to aid in the establishment of improved design specifications for highway and roadside safety are emphasized.

• RESEARCH during the past several years at the Cornell Aeronautical Laboratory (CAL) has led to the development of a computer model representing the dynamics of a motor vehicle in contact with a variety of roadway and roadside terrain features and certain classes of obstacles (1, 2). This research has been sponsored by the Federal Highway Administration as part of an overall research program aimed at the development of improved roadside protective systems to enhance occupant survivability in single motor vehicle collisions.

Rigorous comparisons of simulated responses with full-scale test responses under a variety of maneuvers have substantiated the validity of the model in situations where all major external forces acting on the vehicle are introduced through the tires (1, 3). Preliminary comparisons of collision predictions with experimental results for certain classes of fixed objects also exhibit generally close agreement (4, 5, 6).

The existing capabilities of the computer model can be and are being exploited to converge on improved roadway and roadside geometrics and protective devices. The present version of the model represents, with confidence, the motions and responses of a motor vehicle on the roadway or in contact with the terrain irregularities of the roadside. All major vehicle nonlinearities, detailed braking dynamics, and driver control inputs have been included.

The CAL effort, at full fruition, will provide analytic procedures for evaluating the performance of a variety of roadside structural concepts in their ability to "protect" the vehicle and its occupants in the collision environment.

As a supplement to the conventional time-history form of simulation outputs, an auxiliary computer-graphics program has also been developed within the CAL research program to produce perspective drawings of the simulated vehicle as seen from selected viewing positions and at selected times during a predicted event (7, 8). Figure 1

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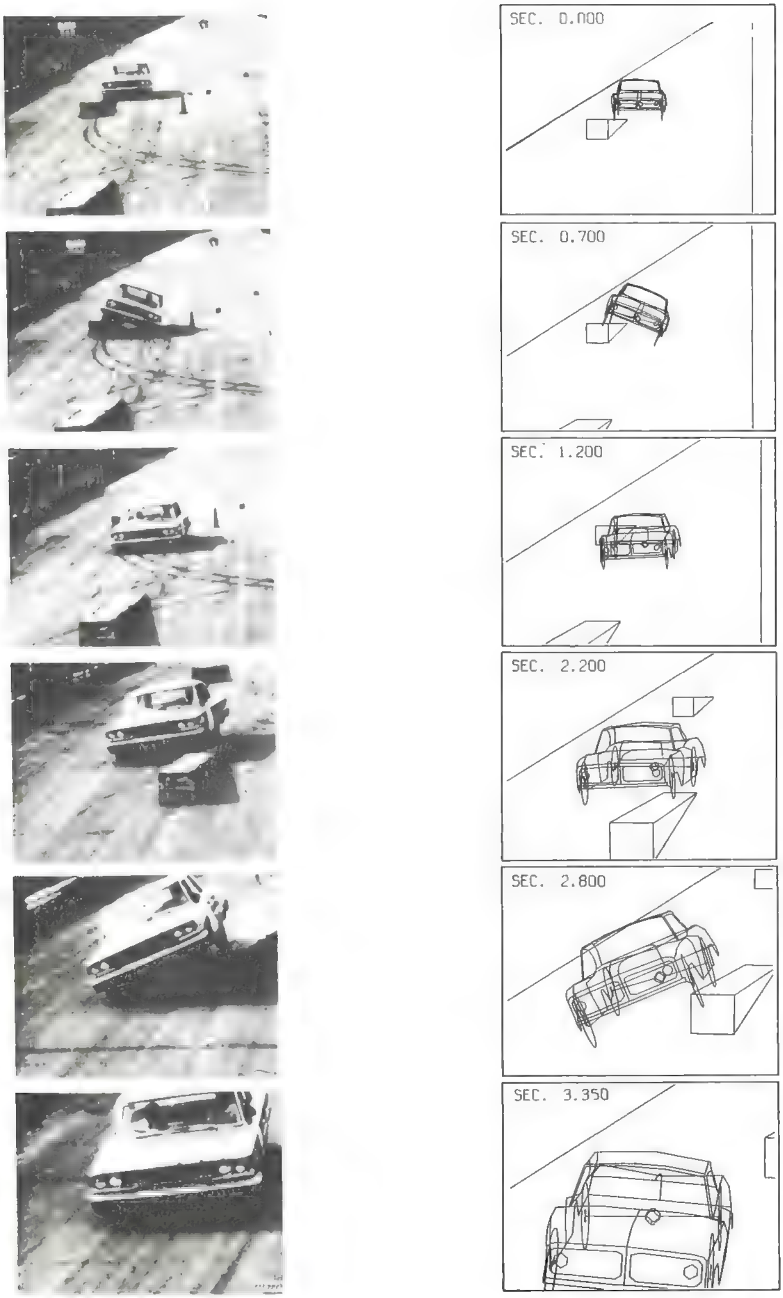

Figure 1. Alternate ramp traversal at 30 mph.

shows the graphics display for a ramp traversal; a series of photographs taken during one of the validation experiments is shown on the left while the corresponding model prediction is shown on the right. Ongoing and future model extensions include improvement in the treatment of collision mechanics, i.e., the representation of the vehicle structural interaction with fixed objects along the roadside and the vehicular responses resulting therefrom.

Evaluation of existing and proposed highway geometrics and protective features by experimental means requires a selection of specific test conditions and procedures to represent the wide distributions of the variables (e.g., vehicle sizes, speeds, angles of approach, and evasive control inputs) that exist in actual single-vehicle collisions. The total number of tests of a given highway feature must be limited, in view of the costs associated with full-scale testing, and the interpretation and extrapolation of test results are made difficult, if not impossible, by the overall complexity of the terrain-obstacle-vehicle-occupant system and by the prevalence of system nonlinearities. An exclusively experimental approach to the problem of improving the performance of a complex physical system is neither scientifically complete nor efficient.

Mathematical modeling of physical systems is, of course, the fundamental methodology of engineering analysis and physical research. The present application is unique in the number and extent of the nonlinearities treated and in the overall complexity of the system.

SIGNIFICANCE OF THE RESEARCH

The model in its present status of development and validation is capable of addressing a host of questions that are of interest to the highway safety community and has implications for new highway construction projects as well as safety improvement programs such as TOPICS.

Typical applications that can now be addressed include the efficacy of (a) roadside terrain details such as the degree of side slope flattening, configuration and placement of drainage channels, and geometric and surface characteristics of curbing; (b) protective structures such as geometric and surface characteristics of rigid redirective barriers (e.g., the New Jersey median barrier design and the GM bridge parapet) and limited classes of guardrail structures; and (c) highway design practices such as geometric and surface interactive elements of horizontal curvature and superelevation and consideration of transition and spiral connections.

It is anticipated that by mid-1971 the development of those aspects of the model associated with interaction between the vehicle and fixed objects will have progressed to the point where greater generality can be accommodated in the treatment of roadside structures.

VALIDATION APPROACH

A high degree of correlation has been achieved between the responses of the vehicle predicted by the computer model and those measured experimentally in a rigorous investigation of validity that included both separate and combined cornering and ride motions (1). A sampling of these response comparisons is shown in Figures 2 and 3.

One of the objectives in this research program has been to apply each item of the vehicle parameters in a directly measurable form. Because it is a common practice in mathematical modeling to define "equivalent" parameters and to adjust the values to achieve correlation with experimental data, it is well to distinguish the approach taken by CAL. A subcontract was let to the Ford Motor Company for measurement of the vehicle parameters, and the results of these measurements have been used directly. The tire properties were measured by the General Motors engineering staff and were also applied directly. Because of the direct use of parameters as measured by disinterested individuals, it should be very clear that the correlation with experimental responses is genuine, involving no "tune-up" of the simulated vehicle properties.

Within the CAL validation program, repeated runs of all but 1 of the 10 test conditions were performed. It is unfortunate that most other experimental efforts that have been performed in relation to vehicle dynamics and to automobile collisions have not

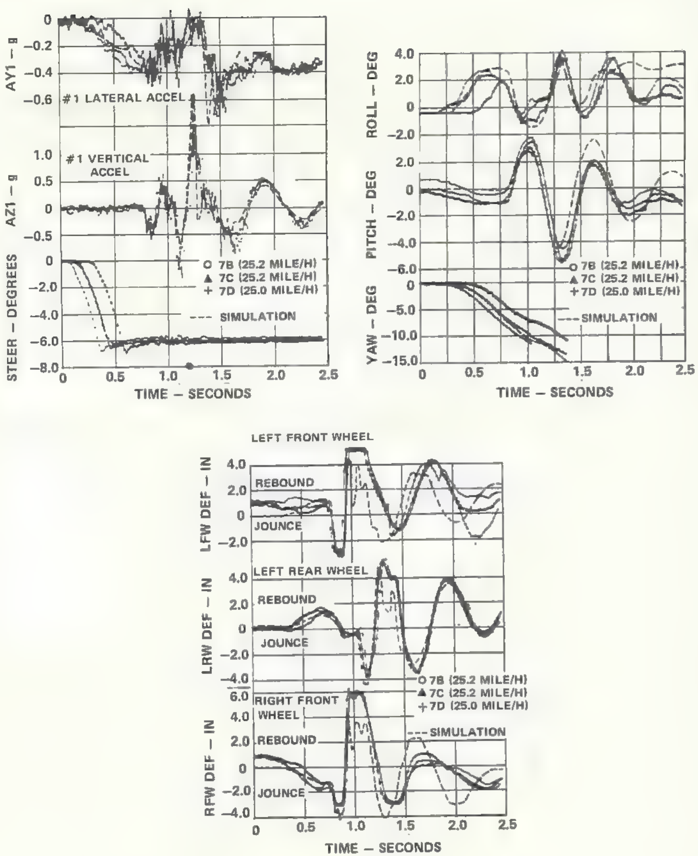

Figure 2. Measured and simulated responses of vehicle traversing 7.1-deg, 6.75-in. high ramp while cornering at 0.4 g.

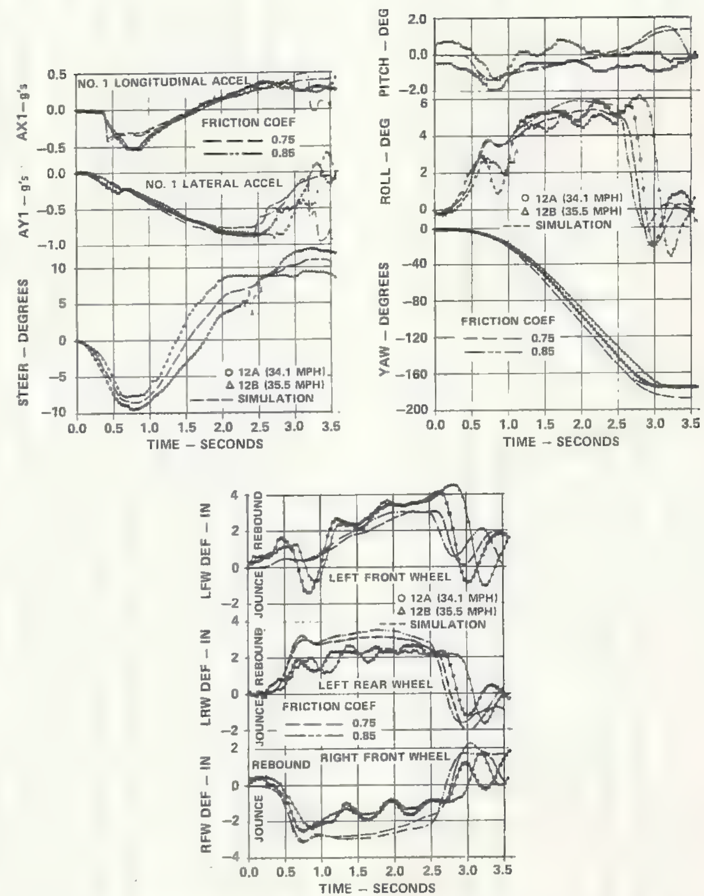

Figure 3. Measured and predicted responses of vehicle in forward skid on dry pavement.

included measures of the variability of the experimental responses. Without such measures, the accuracy of analytical predictions cannot be properly evaluated.

The high degree of repeatability of the experimental responses, shown in Figures 2 and 3, reflects the use of closely controlled experimental procedures. The performance of repeated experiments enables the quantification of statistical reliability and comparison of mean test results with analytic predictions.

The required extent of the total validation effort is determined by the nature of specific intended applications. For example, a primary interest in kinematics (i.e., the trajectory of the vehicle), as opposed to the detailed time history of acceleration, could be satisfied with a less extensive validation effort, in view of the direct measurements that can be made (i.e., no instrumentation response problems) and the "smoothing" or "filtering" effect of double integration of the simulated acceleration. On the other hand, a primary interest in acceleration waveform and peak accelerations requires a greater validation effort.

MODEL DESCRIPTION

The vehicle is treated as an assembly of 4 rigid masses, as shown in Figure 4, with a total of 15 degrees of freedom. These consist of the 6 degrees of freedom associated with 3-dimensional, rigid-body motions of the sprung mass; 5 degrees of freedom associated with motions of the unsprung masses (wheel assemblies) relative to the sprung mass (i.e., suspension deflections and front wheel steer); and 4 rotational degrees of freedom of the wheels. The steer mode degree of freedom, for which any inertial coupling effects are neglected, is introduced at the front wheels when rigid obstacles (e.g., curbs) are encountered by either or both of those wheels. The rotational degrees of freedom of the 4 wheels have been incorporated in order that the effects on tire forces of rotational wheel slip, in braking or traction, can be approximated.

Inertial properties, suspension and tire characteristics, and the detailed brake system are represented. The geometric and surface details of the terrain profile are introduced as an input to the model. Likewise, open-loop control parameters in the form of the steer angle at the front wheels, the throttle setting, the hydraulic pressure

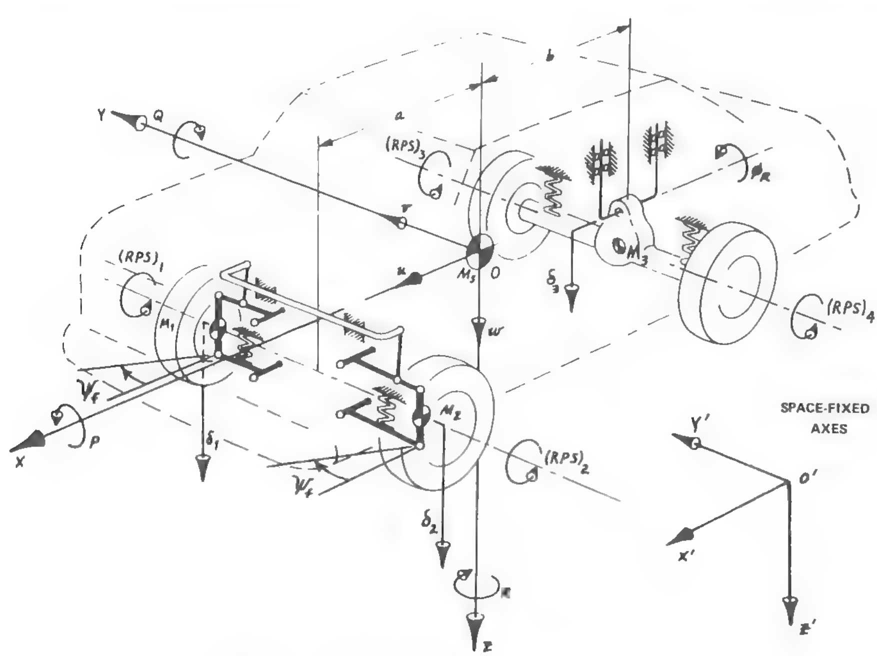

Figure 4. Analytical representation of vehicle.

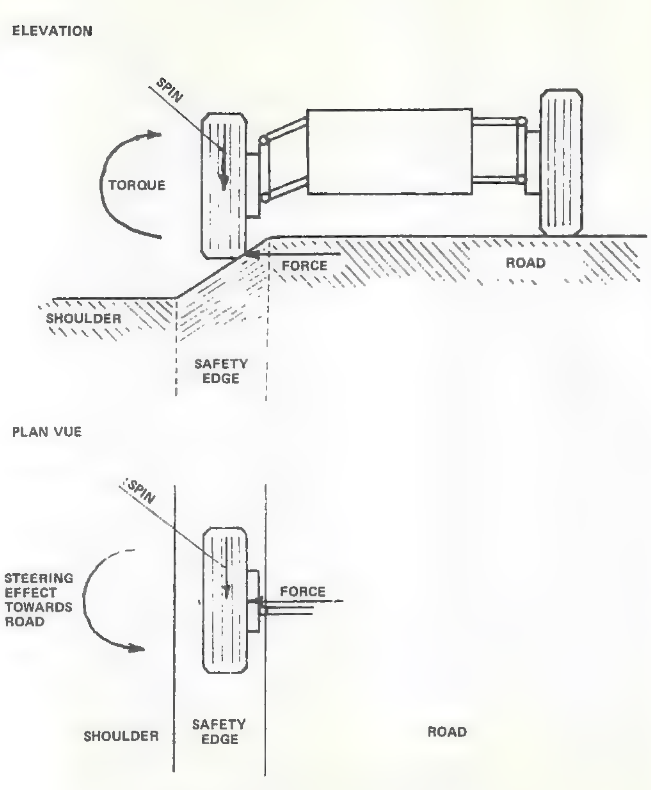

Figure 5. BRIG Road Safety Edge.

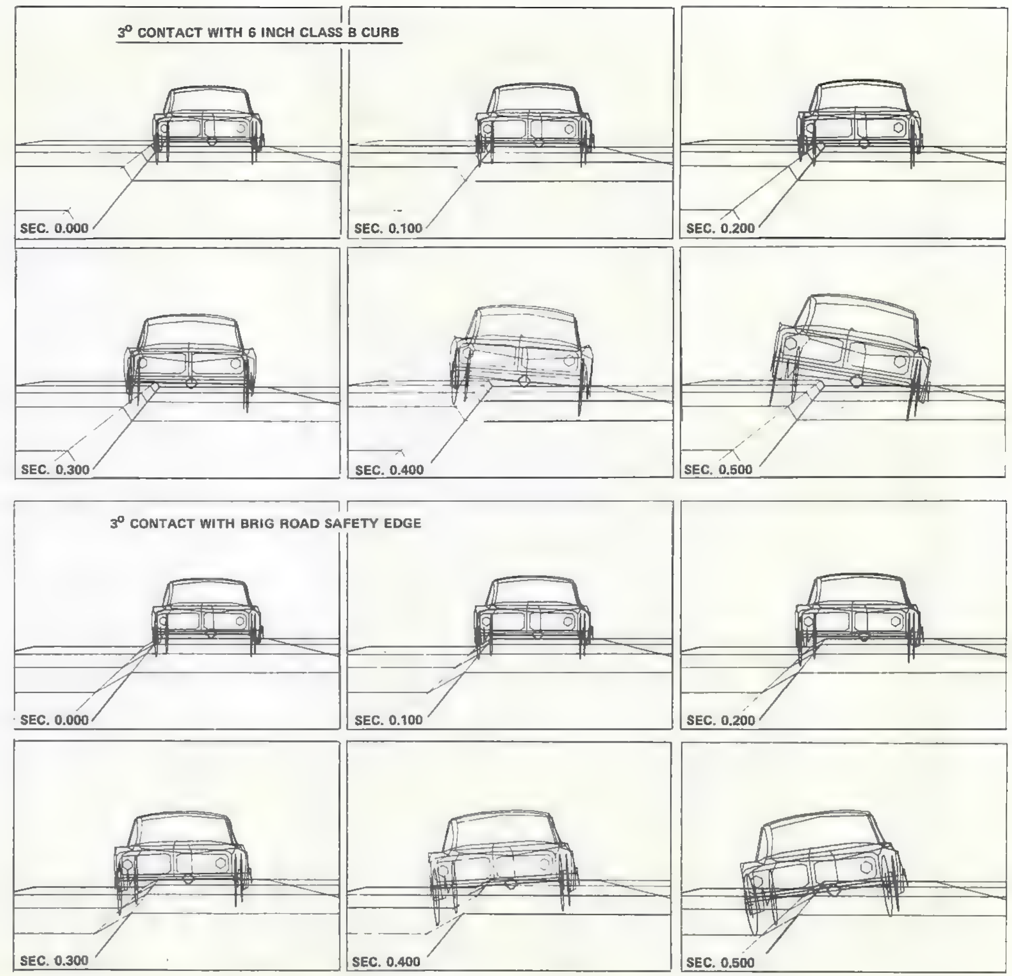

Figure 6. Computer-graphics display of simulated 60 mph.

in the brake system, and the transmission ratio are entered as tabular functions of time. There is also a provision within the model to simulate closed-loop (driver) control in the form of inputs.

SAMPLE APPLICATIONS

The following sample applications are presented to demonstrate the versatility of the described analytical aid.

BRIG Road Safety Edge Concept

The BRIG Road Safety Edge concept (9), shown in Figure 5, consists of a depressed edge between the pavement and the roadside shoulder. It is claimed that this device produces corrective steer effects through the mechanism of gyroscopic precession in the steered wheels of an automobile departing from the pavement.

The BPR-CAL computer simulation program was applied in a preliminary evaluation of the BRIG concept (Fig. 6). It was concluded that the proposed road edge will produce corrective steer effects only under extremely limited conditions of departure from the pavement and that the mechanism through which the effects are produced differs substantially from that described in the related U.S. patent and in an earlier analysis supporting the patent claims. In particular, it was concluded that the role of gyroscopic precession in influencing automobile steering responses during contacts with either conventional curbs or the BRIG Road Safety Edge is a secondary role that can be neglected in first-approximation calculations and that the redirective properties of the BRIG Road Safety Edge are effective only under conditions of extremely gradual departures from the roadway.

Torsion-Post Bridge Rail

The capability of the BPR-CAL computer model for simulating vehicle impacts with roadside structures was used in a recent application of the simulation in a research program to evaluate a new type of bridge railing conceived by the District of Columbia Department of Highways and Traffic (6). A primary feature of the concept was the use of torsion posts for supporting the rails to provide flexibility of the structure and to absorb and subsequently release energy transferred from an impacting vehicle by operating within the elastic range of the post material. It was hypothesized that good redirection characteristics might result from a configuration having a larger inertia that would effectively delay release of the energy stored in the torsion posts until the vehicle had already departed from the impacted area.

The computer simulation was used to predict the dynamic responses of the vehicle and bridge railing under various impact conditions and to provide guidance in the selection and design of prototype torsion-post bridge railing configurations for subsequent full-scale experimental testing.

Sequence photos of a test of one of the experimental bridge railings are shown in Figure 7. Figure 8 shows a comparison of the predicted and measured responses of the vehicle and bridge rail for the test impact conditions of 52 mph and 25-deg angle of approach.

Bridge Parapet Investigation

The computer simulation was applied in a brief investigation to determine how the response of a vehicle impacting a rigid, redirecting barrier such as the General Motors bridge parapet or the New Jersey median barrier would be affected by variations of the friction coefficient of the sloped face (10). Simulation runs of vehicles impacting at 30 mph and 5-deg angle of approach and 50 mph and 12 deg for values of friction coefficient between 0.25 and 0.7 were made. A representative graphic display of the predicted response of a vehicle for comparison with photos made during a test performed by General Motors (11) is shown in Figure 9. The results of the study substantiated the conclusion previously reported by Lundstrom et al. (11) that the value of the friction coefficient of

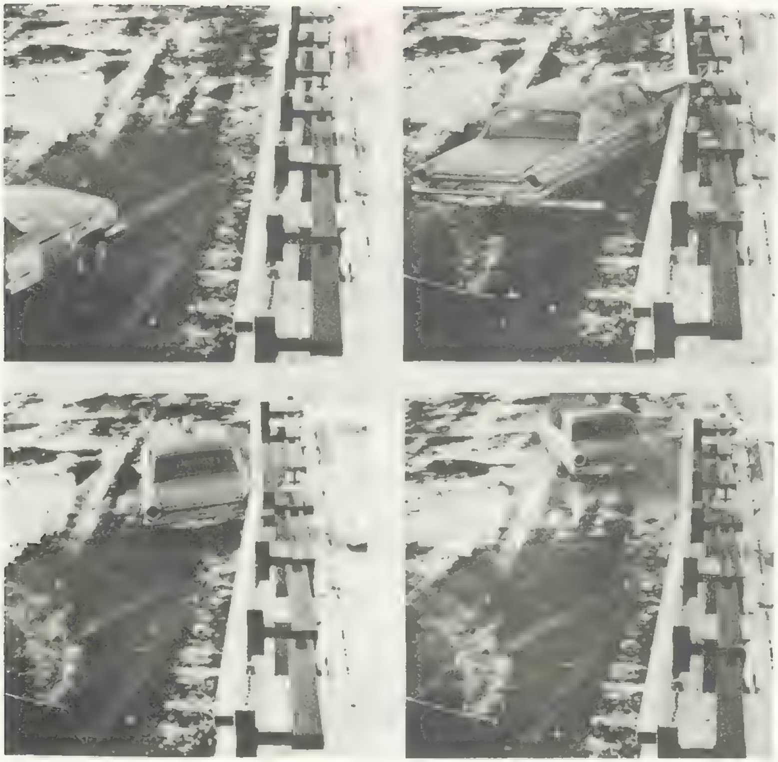

Figure 7. Sequence of test 1 impact.

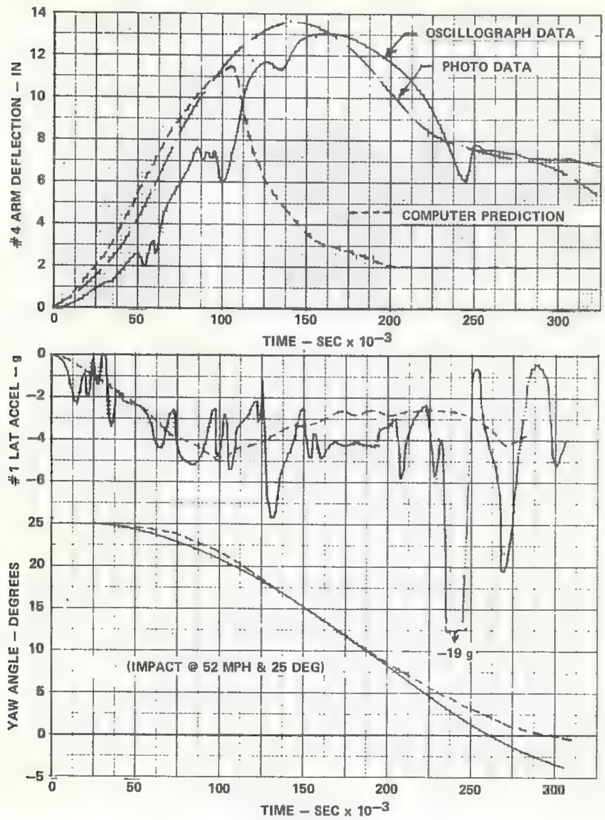

Figure 8. Vehicle and bridge railing responses in test 1.

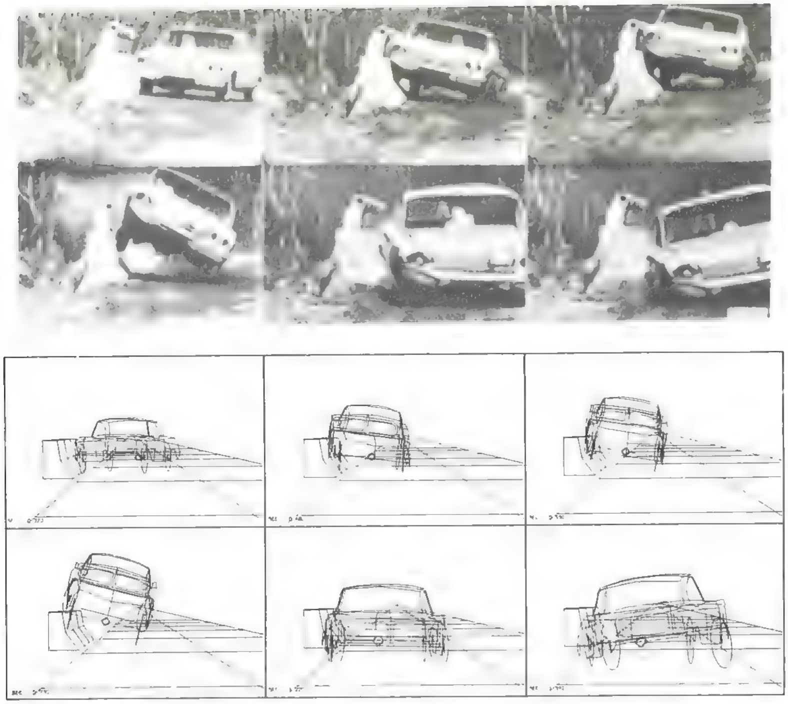

Figure 9. Experimental and predicted vehicle responses for 50-mph, 12-deg collision with General Motors bridge parapet.

the surface does not significantly affect the response of the vehicle or the height of climb up the wall.

Railroad Grade-Crossing Topography

An exploratory analytical investigation was conducted to determine speed-topography combinations for which vehicle responses when traversing railroad grade crossings may be sufficiently violent to induce loss of control (12).

It was found that neither steer effects from tire contacts with rails protruding above the highway surface nor "wheel hop" excitations from protruding rails or other terrain irregularities constitute a significant control problem at grade crossings. Crossings situated on crest vertical curves were concluded to be potentially prone to loss of vehicle control, contingent, of course, on the specifics of the cross section. The exploratory analysis demonstrated most importantly that dimensional tolerances for highway-railroad grade crossings may be objectively defined on the basis of dynamic vehicular responses assessed through the BPR-CAL model for a representative family of highway vehicles.

Figure 10 shows a computer-graphics display of a simulation run for an actual railroad crossing with a crest vertical curve in the Buffalo, New York, area where loss of vehicle control resulted in a ran-off-roadway fatal accident. Potential loss of control effects were exaggerated by the selection of a speed of 70 mph, and a steer input of 3 deg (at the front wheels) was entered while the front wheels were off the ground.

In the drawings shown in Figure 10, the simulated event is viewed from a constant distance ahead of the vehicle and along the centerline of the road. The simulated vehicle went up on its 2 right wheels and then departed from the opposite side of the roadway. Such a driver maneuver conceivably could have been made in an attempt to avoid encroaching on the ditch on the right side of the roadway, with an excessive steer input applied while the front wheels are off the ground.

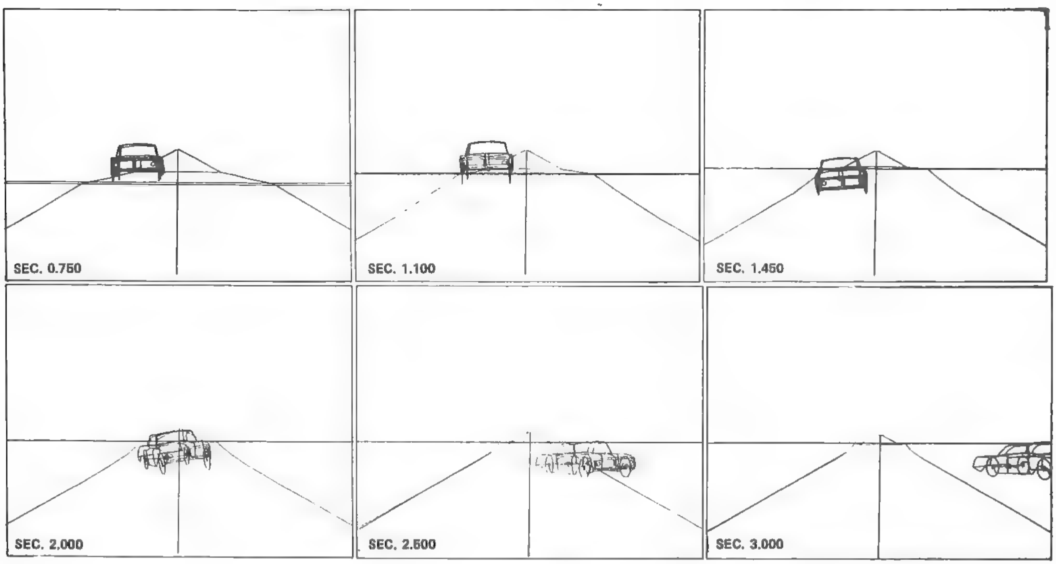

Figure 10. Simulated traversal of railroad crossing.

CONCLUDING REMARKS

This paper has presented a brief review of the status and capabilities of the BPR-CAL computer simulation of single vehicle accidents. The selected approach in computer implementation for this analytical aid (i.e., digital in the Fortran IV language) has been aimed at ease of transfer of the computer program to other research facilities. The objective of FHWA in this selection has been to contribute to a general elevation of the state of the art of highway vehicle dynamics by making the computer program readily available. To date, copies of the computer program have been distributed to 25 research organizations.

The potential of the model as an aid for objective evaluation of highway design practices and for rational development of warrants and specifications for roadside safety is virtually untapped. The capability to represent a cross section of vehicle and driver evasive maneuvers affords a unique opportunity for definitive analyses.

The model also has provisions for the generation of realistic closed-loop driver control inputs (13) and for detailed treatment of antilock braking systems (2). It also has an important potential as an aid in the analytical reconstruction of highway accidents.

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ALIGNMENT COORDINATION IN HIGHWAY DESIGN

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The purpose of this research was to study the coordination of horizontal and vertical elements of highway alignment in order to provide some definitive guidelines for the highway designer. The technique of plotter-drawn perspectives was used in this study. A battery of computer programs was developed that provided a means of rapidly and economically producing perspective drawings of a proposed or existing roadway. The programs provided for the computation of highway alignment and space coordinates, the perspective picture plane coordinates, and a "plot" program, all with the observer (eye location) at any arbitrarily selected location. The capabilities of the computer program are described in detail. A large number of perspective drawings were made for a detailed study of the coordination of vertical and horizontal curves as well as for the study of the visual appearance of spiral curves.

• THE PURPOSE of the research described in this report was to study the coordination of horizontal and vertical elements of highway alignment in order to provide some definitive guidelines for the highway designer.

Cron (1), Pushkarev (2), and Smith (3) speak of problems of fitting the highway to the landscape, of the visual quality of the highway, and of the visual discontinuities that have occurred in many of the existing highways. They all point to a common problem: How can the designer "see" his 2-dimensional (plan-profile) design as a 3-dimensional finished product? Recent research by Smith and Yotter (4, 5) has shown conclusively that electronic plotter-drawn perspectives of the roadway are a highly versatile and valuable tool in attacking the problem of actually accomplishing good visual design.

In view of the experience with plotter-drawn perspectives, the authors decided to use the technique to accomplish the objectives of this research. A battery of computer programs was developed that provided a means of rapidly and economically producing perspective drawings of a proposed or existing roadway. The programs provided for the computation of highway alignment and space coordinates, the perspective picture plane coordinates, and a "plot" program, all with the observer (eye location) at any arbitrarily selected location. The capabilities of the computer program are described in detail in another report (6).

COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENT

One of the well-recognized visual-design problems is that of combining horizontal and vertical curves so they present an alignment that is smoothly flowing visually. It is nearly impossible for the designer to visualize the real 3-dimensional curve by studying the horizontal and vertical alignments as they are shown on the plan-profile sheets. In order to aid the designer in his task, this study was made by using perspective drawings of various combinations of sag vertical curves and horizontal curves.

The geometry of the study alignment was as follows: Single roadway, 24 ft in width; and observer position, 4 ft left of roadway centerline and 3.5 ft above roadway surface.

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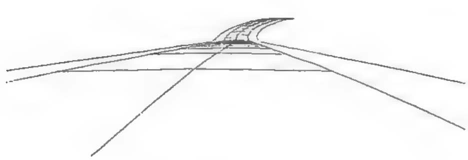
The sighting distance, S, from the observer to the midpoint of the vertical curve varied. The total change in horizontal direction, Δ, was 3 deg 14 min right. For the vertical alignment the grade of the back tangent, g_1, was -0.77 percent, and the forward tangent grade, g_2, was +1.53 percent, giving a total change of grade, A, of 2.30 percent.

The effects of the displacement of PI's of horizontal and vertical curves, length of curve relationships, and observer position were studied by varying these elements as follows:

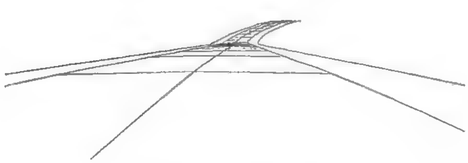
1. The PI's coincided, the vertical curve PI was located 250 ft ahead of (prior to) the midpoint of the horizontal curve, and the vertical curve PI was located 500 ft ahead of the midpoint of the horizontal curve;
2. Lengths of horizontal curve of 970, 2,000, and 3,000 ft were each combined with vertical curve lengths of 800, 1,000, 1,500, and 2,000 ft; and
3. For each combination of PI separation and curve length, the observer's location was set so that sighting distances of 2,400, 1,900, 1,400, and 900 ft were obtained (S was taken as the distance from observer to the PI of the vertical curve).

Figures 1 through 9 are perspectives selected from the 132 drawn for this particular study.

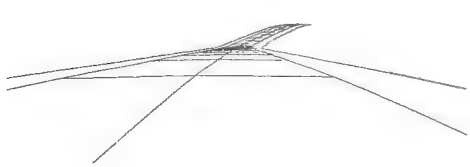
Figure 1 shows perspectives with a sight distance of 2,400 ft; the length of horizontal curve, L_H, is 970 ft, and the length of vertical curve, L_V, is 800 ft. Figure 1a shows a PI separation of 500 ft (PVI is 500 ft nearer the observer than is the PI of the horizontal curve). In this case the PC of the horizontal curve and the PVI nearly coincide. According to Pushkarev (2), "The vertical alignment is shifted half a phase with respect to the horizontal alignment." In this case the beginning of the vertical curve precedes or leads the horizontal curve by about 415 ft. There is a sharp break or discontinuity in the left edge of the roadway and in the trace of the centerline. Figure 1b shows a PI separation of 250 ft, and the alignment is shifted about $\frac{1}{4}$ phase. The vertical curve leads the horizontal curve about 165 ft. There is a sharp break in the left edge of the roadway although not so sharp as that shown in Figure 1a. Figure 1c shows the same 2 curves with the PI's coinciding. These curves are "in phase." In this case the horizontal curve leads the vertical curve by 85 ft or about 10 percent of the curve length. There is a marked improvement in the smoothness of the left edge of the roadway.



A. No. 17101 PI-Sep. = 500'



B. No. 17001 PI-Sep. = 250'



C. No. 17201 PI-Sep. = 0'

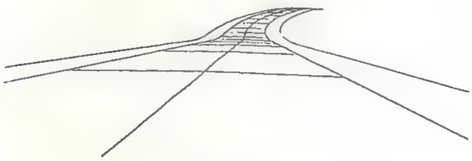
Figure 1. Perspectives for various PI separations, L_H = 970 ft, L_V = 800 ft, and S = 2,400 ft.

Figure 2 shows the same curve as that shown in Figure 1a, but the observer is located at different sight distances. As the observer approaches the curve there is a noticeable smoothing of the left edge of the roadway. Figure 2d shows that, where the observer is only 900 ft from the PVI, the roadway appears quite smooth and flowing.

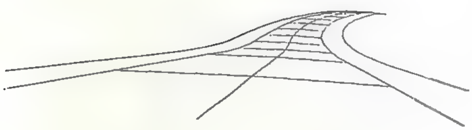
Figure 3 shows perspectives with S = 2,400 ft, L_H = 2,000 ft, and L_V = 800 ft and for PI separations of 500, 250, and 0 ft. All these perspectives show an apparent "hump" or slight inflection particularly in the left edge of the roadway. The centerlines show some discontinuity. The inflection shown in Figure 3a is somewhat less, and this is explained by the fact that the horizontal curve leads the vertical by 100 ft. The inflection shown in Figure 3c is due to the fact that the vertical curve is too short as compared to

A. No. 17101 S = 2400'

B. No. 17102 S = 1900'



C. No. 17103 S = 1400'

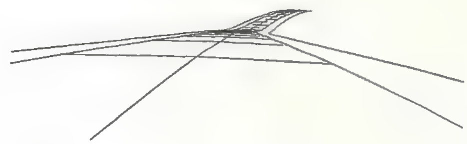


D. No. 17104 S = 900'

Figure 2. Perspectives for various sighting distances L_H = 970 ft, L_V = 800 ft, and PI separation = 500 ft.

A. No. 17105 PI-Sep. = 500'

B. No. 17005 PI-Sep. = 250'



C. No. 17205 PI-Sep. = 0'

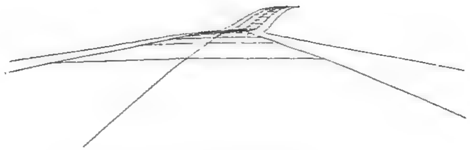
Figure 3. Perspectives for various PI separations, L_H = 2,000 ft, L_V = 800 ft, and S = 2,400 ft.

the horizontal curve. If one were to approach the curve shown in Figure 3a from the right, an inflection similar to that shown in Figure 3c would be apparent.

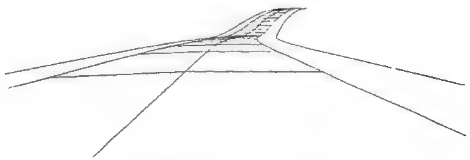
Figure 4 shows a 3,000-ft (L_H = 3,000) horizontal curve and an 800-ft vertical curve with PI's coincident. The obvious inflection of the left edge of the roadway does not disappear until the observer is 900 ft from the PVI. The apparent inflection or reverse curvature is due to the fact that the vertical curve is much too short when compared to the horizontal curve. No amount of PI movement will solve this problem.

Figure 5 shows conditions quite similar to those shown in Figure 1. In this case the vertical curve, L_V = 1,000, is 30 ft longer than the horizontal curve. Figures 5a and 5b show curves shifted about $\frac{1}{2}$ and $\frac{1}{4}$ phases respectively and indicate very clearly that the vertical curve must not significantly precede the horizontal curve. Figure 5c shows that, where the vertical curve leads the horizontal curve by only 15 ft, no visual discontinuity is apparent.

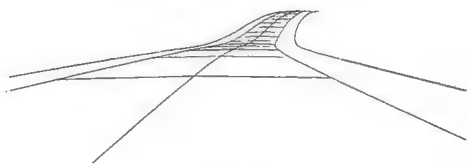
Figure 6 shows perspectives with S = 2,400 ft, L_H = 970 ft, and L_V = 1,500 ft. The PI separations are 500, 250, and 0 ft. The inflection or discontinuity are shown in the left edges of the roadway in Figures 6a and 6b. The authors believe this is due to the fact that the vertical curve leads the horizontal curve in each case. A rather disconcerting discontinuity along the left edge of roadway is shown in Figure 6c, a case in which the PI's coincide. This can only be attributed to the fact that the vertical curve leads the horizontal or that, in this case, the vertical curve is significantly longer than the horizontal curve.



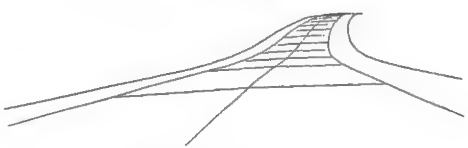
A. No. 17209 S = 2400'



B. No. 17210 S = 1900'

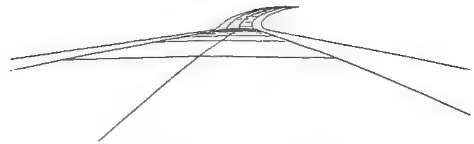


C. No. 17211 S = 1400'

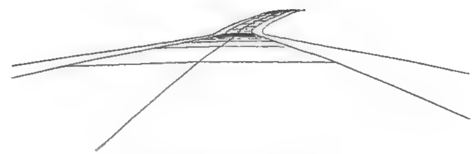


D. No. 17212 S = 900'

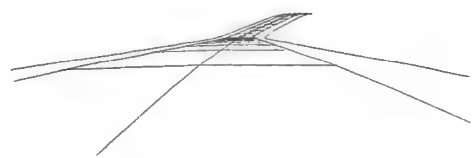
Figure 4. Perspectives for various sighting distances, L_H = 3,000 ft, L_V = 800 ft, and PI separation = 0 ft.



A. No. 17113 PI-Sep. = 500'

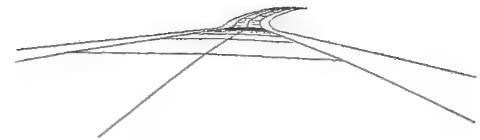


B. No. 17013 PI-Sep. = 250'

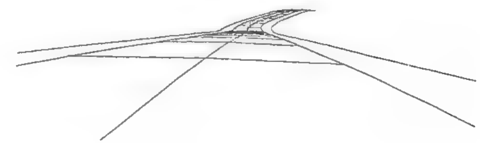


C. No. 17213 PI-Sep. = 0'

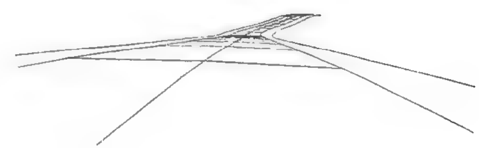
Figure 5. Perspectives for various PI separations, L_H = 970 ft, L_V = 1,000 ft, and S = 2,400 ft.



A. No. 17125 PI-Sep. = 500'

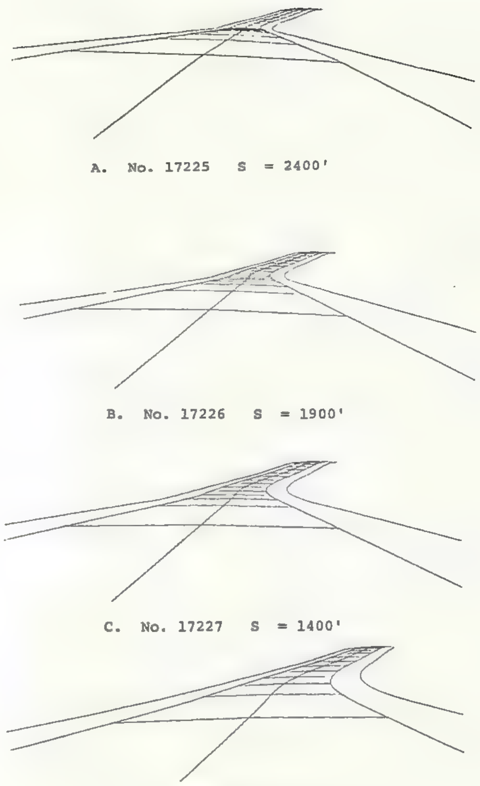


B. No. 17025 PI-Sep. = 250'



C. No. 17225 PI-Sep. = 0'

Figure 6. Perspectives for various PI separations, L_H = 970 ft, L_V = 1,500 ft, and S = 2,400 ft.



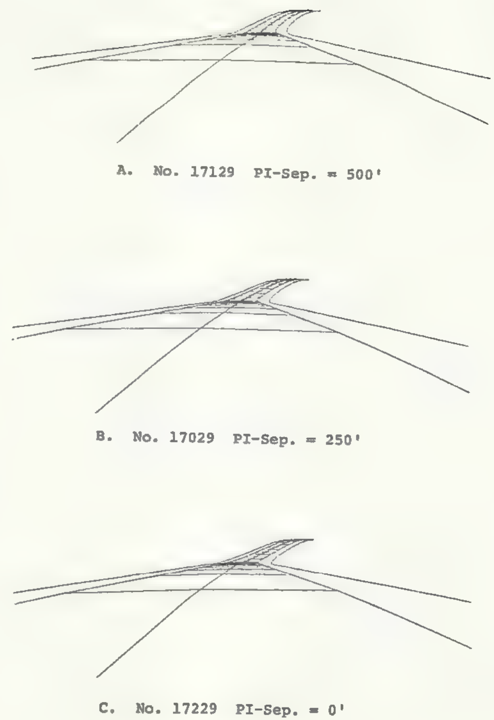
A. No. 17225 S = 2400'

B. No. 17226 S = 1900'

C. No. 17227 S = 1400'

D. No. 17228 S = 900'

Figure 7. Perspectives for various sighting distances, L_H = 970 ft, and L_V = 1,500 ft, and PI separation = 0 ft.

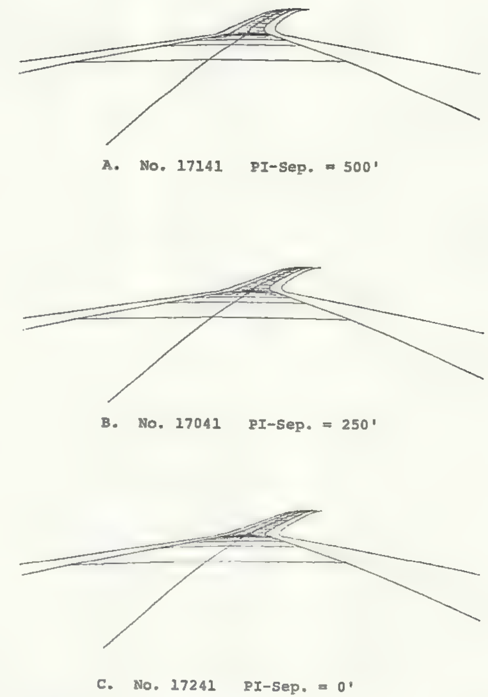


A. No. 17129 PI-Sep. = 500'

B. No. 17029 PI-Sep. = 250'

C. No. 17229 PI-Sep. = 0'

Figure 8. Perspectives for various PI separations, L_H = 2,000 ft, L_V = 1,500 ft, and S = 2,400 ft.



A. No. 17141 PI-Sep. = 500'

B. No. 17041 PI-Sep. = 250'

C. No. 17241 PI-Sep. = 0'

Figure 9. Perspectives for various PI separations, L_H = 2,000 ft, L_V = 2,000 ft, and S = 2,400 ft.

Figure 7 shows the same curve with the different sight distances. Figure 7d shows that, even when the observer is only 150 ft from the beginning of the vertical curve, the centerline looks a bit awkward.

Figure 8 shows perspectives with S = 2,400 ft, L_H = 2,000 ft, and L_V = 1,500 ft. There is very little apparent difference among the various PI separations shown. There is, however, a subtle inflection or break in the left edge of the roadway. The authors conclude, therefore, that the 1,500-ft vertical curve is a bit too short relative to the length of horizontal curve.

Figure 9 shows perspectives with S = 2,400 ft, L_H = 2,000 ft, and L_V = 2,000 ft. There is little apparent difference among the PI separations shown. The authors would not be surprised if, for longer sight distances, inflections became seriously apparent for those shown in Figures 9a and 9b. It is apparent that, if one uses very long horizontal and vertical curves, the visual problems caused by poor coordination are less serious.

The following conclusions were drawn from this study and are listed as rules for the coordination of vertical and horizontal alignment. In order for a roadway to have a smooth flowing appearance (which is also described as an absence of inflections or visual discontinuities), (a) the PI's must nearly coincide (within about 10 percent of horizontal curve lengths); (b) the length of vertical and horizontal curves must be nearly equal (within about 10 percent); and (c) if the curves are not of the same length, the horizontal curve should "lead" or slightly precede the vertical curve, i.e., the vertical curve should never be significantly longer than the horizontal curve.

The following observations were also made: (a) As the curves become longer (more generous alignment), slight violations of rules 1 and 2 are less serious; (b) the farther the observer is from the curves (longer sighting distances), the more accentuated are the visual discontinuities; and (c) if the rules are violated, it is strongly recommended that a series of roadway perspectives be drawn so that any visual discontinuities can be detected. These rules agree very closely with those suggested by Pushkarev (2, pp. 197-199).

SPIRAL PROBLEM

The problem of the foreshortened appearance of horizontal curves was studied. In particular, an attempt was made to determine criteria for the selection of a spiral length that would ensure a smooth and flowing appearance in the ribbon of roadway. Additional detailed information on this portion of the study is given in another report (6).

From personal observations and the information gained from the literature search, it was hypothesized that the factors affecting the visual appearance of a horizontal curve were observer distance, OD, display angle, DA, and the geometry of the curve. Observer distance in this case is defined as the distance from the observer to the beginning of the curve, PC or TS, whichever is appropriate; display angle is defined as the angle between the observer's line of sight and the plane containing the curve; and the geometry of the curve is defined as the spiral length, if any, and the limiting radius of the curve.

Approximately 500 perspective pictures were plotted with varying sight distances, limiting degree of curve, spiral length, and display angle. For any given set of geometric conditions, the sight distance and display angle were varied by moving the observer position. Moving the observer up into space above the roadway created artificial display angles and simulated the case where a sag vertical curve is located between the observer and the horizontal curve without the disruptive element caused by the vertical curve.

A wide range of geometric conditions was investigated. The degree of curve was varied from 30 min to 5 deg. Some of the curves were constructed without a spiral transition so that the visual effect of adding a spiral curve could be tested. The spiral lengths varied from 100 to 2,000 ft. The lengths of spirals examined at each limiting degree of curve are as follows:

0.5 Deg	1 Deg	2 Deg	3 Deg	4 Deg	5 Deg
0	0	0	0	0	0
500	250	300	200	200	100
2,000	500	500	600	500	400
	1,000				
	2,000				

All curves were rated according to their smoothness of appearance and the rate at which they seemed to diverge from the tangent. The rating scale was acceptable, questionable, or unacceptable. Although this may seem a crude rating scale, it was felt that any refinement beyond this level was not justified because at some point the decision had to be made whether a curve was acceptable or unacceptable.

The following was concluded from the spiral curve study:

1. Spiral curves do improve the appearance of most circular curves;
2. When the observer is near the plane of the curve, there is no significant difference in the appearance of a spiraled and an unspiraled curve;
3. As the distance from the curve to the observer increases, the length of spiral needed increases for good visual quality;
4. As the height of the observer raises above the plane of the curve, increasing display angle, the length of spiral needed decreases for good visual quality;
5. Curves that consist entirely of spiral curves give the best visual appearance, all other conditions being equal; and
6. The rate at which a curve visually appears to diverge from the tangent affects the visual quality of the curve.

ACKNOWLEDGMENTS

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EVALUATION OF GEOMETRIC IMPROVEMENTS MADE UNDER TOPICS

Julie Anna Cirillo Fee, Federal Highway Administration; and
Neilon J. Rowan, Texas Transportation Institute

This paper summarizes statements made at a conference session held during the 50th HRB Annual Meeting and also remarks made during the discussion at the conclusion of the session. An addendum details specifics of TOPICS as of September 1, 1971.

INTRODUCTION

Neilon J. Rowan, Texas Transportation Institute

Conference sessions are designed to satisfy several basic purposes of communication. In one case, a conference session can be a meeting of researchers in which ideas and experiences in research techniques are exchanged, and all researchers benefit from their attendance. Also, a fruitful conference session can be realized by bringing together researchers and practicing engineers to discuss common interests. The practicing engineer benefits from additional knowledge gained from the researcher, while the researcher gains a better understanding of the problems of the practicing engineer. Thus, each receives a better insight toward accomplishing his own objectives. All go away enhanced for having participated in the conference session. It is this latter type of session that we tried to achieve in bringing together representatives of research agencies, municipalities, states, and federal agencies associated with the Traffic Operations Program to Increase Capacity and Safety(TOPICS).

The subject of this session is a most timely one because it is an organized effort of federal cooperation with the urban community. The Federal Highway Administrator stated in an address that the federal government must be an active partner in the urban community. He indicated, in general, that we must learn to live with the automobile, the bus, the truck, and the roadway network. Mobility will continue to be a community goal, and a common goal for all concerned with transportation.

TOPICS—CONCEPT, IMPLEMENTATION, AND ASSESSMENT

Charles J. Keese, Texas Transportation Institute

The Federal-Aid Highway Act of 1968 first authorized the Traffic Operations Program to Increase Capacity and Safety. The main thrust of TOPICS activities has been aimed at relieving existing traffic bottlenecks in urban areas. Implementation of TOPICS required a cooperative effort among city, state, and federal governments. The initial Policy and Procedure Memorandum 21-18 was issued by the Federal Highway Administration January 17, 1969, and required the following items to be fulfilled during participation in the program:

1. There must be full coordination of TOPICS with a comprehensive transportation planning process;
2. TOPICS must be compatible with a general plan of long-range street and highway improvements;
3. There must be an area-wide TOPICS plan normally requiring no more than 5 years for implementation and based on the comprehensive transportation plan;

Sponsored by Committee on Operational Effects of Geometrics and presented at the 50th Annual Meeting.

4. Formal approval of a primary Type II system of streets (i.e., extension of primary and secondary road systems into urban areas) must be obtained from the FHWA division engineer; and

5. Priorities must be set for TOPICS projects to ensure that the most pressing needs are met first.

By the end of 1969 it became obvious that the program was not being advanced. Some reasons for this are as follows:

1. Development of state policies and procedures required time;
2. Staffing of technical positions was necessary and required time;
3. Insufficient lead time was given to include matching funds in the 1969 state and local budgets;
4. Although there was a backlog of highway improvement needs, there was no substantial backlog of construction projects (plans, specifications, and estimates) awaiting implementation, and their preparation takes an average of 1 to 2 years; and
5. Federal policies and procedures were not considered to be conducive to an immediate-action construction program.

To alleviate these problems, Policy and Procedure Memorandum 21-18 was reissued on May 28, 1970. The first significant change was the elimination of requirement 2. This requirement had been causing extra work, delay, and confusion because of ambiguity of the definition of "general plan."

The second major change was a redirection in emphasis for the area-wide TOPICS plan. Many states and cities were spending a large amount of time and money in preparing study reports, thus delaying the actual implementation of known needed improvements. As a result of this change, individual projects were justified by information extracted from the traffic department's normal functions and aided in furthering the objectives of TOPICS.

Assessing the value of TOPICS is a matter that has not received a great amount of attention. Normally, assessment is associated with an evaluation of improvements after completion of the project, and this is a most important aspect. However, assessment begins with planning the program. Development of criteria for selecting improvements should involve an orderly process of assessing the relative value of various alternative improvements in terms of cost effectiveness. In order that the selection criteria be most realistic, evaluation of early TOPICS projects should be given a very high degree of attention; and every effort should be made to provide feedback from the evaluation to the planning process in the closest time span. This should involve the greatest cooperation of federal, state, and city interests if we are to realize the maximum return from our tax dollars and technological efforts.

CRITERIA FOR SELECTION OF GEOMETRIC IMPROVEMENTS UNDER TOPICS

David G. Snider, Desmoines Traffic and Transportation Department

During fiscal year 1971 Iowa received \$2.1 million in TOPICS funds as its initial share. Des Moines received \$301,680 per year for the first 2 years. In Iowa, TOPICS funds can be spent only for construction, not for planning. By January 1971, Des Moines had one complete project and stage 1 of the second project under contract.

Project 1 was the channelization and signalization of 2 major intersections. These intersections were a part of a city street being widened from 20 to 50 ft for a 24-block length. Stage 1 of the second project was widening a bridge. Stage 2 will be the channelization and signalization of the intersection located at the immediate north end of the bridge.

The third project will be a complex channelization and signalization of 2 adjacent intersections in a commercial shopping center area. Another project will require installation of 2 pedestrian overpasses at grade schools. Additional projects planned will be channelization and signalization at isolated intersections to be coordinated with city street-widening projects. A rebuilding of the CBD signal system is also scheduled.

Six items were used in developing criteria for selection of projects: accidents, capacity, surrounding land use, system analysis, future growth projections, and variety as to type of projects.

Accident data, per se, constitute a controversial tool for the traffic engineer to use for substantiating any project. However, accidents do give an indication of a situation that needs correction and, more importantly, provide insight into situations to avoid in designing new projects.

Capacity is a major criteria used in project selection. Capacity here is used both in terms of cars per hour per lane and in terms of driver comfort and ease of driving. The higher the level of service is, the more the driver will enjoy his trip because of less strain and ease of driving decisions. (Level of service is measured principally by speed and volume. Capacity is merely a low level of service, level E, but it is the point where maximum traffic flow occurs. The reason for attempting to increase capacity is to reduce congestion and increase flow.) This situation contributes to reduced accident potential as much as good geometric design.

Land use consideration was used to provide a futuristic outlook for an improvement. Existing zoning, as well as trends of zoning in the area, provided a means of making decisions on possible right-of-way needs, additional traffic lanes, expansion of traffic signal phasing, and possible level of service expected in the immediate future as well as in 1990.

The criteria of system analysis was specifically directed at providing spot improvements on streets scheduled for widening throughout their length. Because projects of intersection improvements were selected in continuous street-widening proposals, spots were improved in advance of the total widening, thus providing the needed capacity at major intersections before the capacity of the completed street widening was available.

Future growth projections were used to determine total number of vehicles to be served in the future and to determine traffic volume increases within the next 5 years.

Problem areas, other than the common intersection improvement, were suggested as possible TOPICS projects. This variety of suggestions was also included in the criteria as a means of establishing guidelines and precedents for future projects.

After the projects were selected, 4 basic areas were used to determine an order of priority for the projects. These areas included comparison to the general transportation plan, accident analysis, immediate land use and its potential, and the immediate capacity problem.

The philosophy of the city of Des Moines is to apply sound traffic engineering principles to problem locations and to take local knowledge into consideration. In so doing, improvements will follow a logical priority for all street and highway programs, including TOPICS.

EXPERIENCE IN EVALUATING THE EFFECTIVENESS OF MINOR IMPROVEMENT PROGRAM

Joseph A. Mickes, Missouri State Highway Commission

The 120 Program in Missouri is a spot-improvement program designed to improve with traffic engineering techniques 120 high-accident locations in a 12-month period.

The 120 Program is designed to place a portion of the day-to-day activities of traffic operations engineers into a controlled atmosphere of before-and-after evaluation. Engineers involved in traffic engineering activities daily perform traffic engineering improvements based in some cases on merely a windshield survey in which their judgment and past experience weigh very heavily. Also, there are many minor improvements that are implemented based on the basic study techniques of the traffic engineering profession. These techniques involve volume studies; speed studies; gap studies; evaluation of physical features such as alignment, sight distance, and geometrics; observation studies; and conflict studies.

The 120 Program improvements are divided in a geographic distribution according to administrative districts. The improvements made under the 120 Program are limited to $1,000 per location. The purpose of placing a financial limitation is basically twofold: (a) to eliminate the tendency of looking at a high-accident location and of

developing solutions of major roadway or interchange construction and to encourage use of signs, signals, and markings and (b) to encourage implementation of the improvements within a reasonably short period of time. The results received have been beyond expectations. They verify the notion that you can do a lot of good with a "bucket of paint and couple of signs."

The actual mechanics of the 120 Program are as follows:

1. Each quarter the headquarters office provides collision diagrams, accident rates, and hard copies of all accident reports on 5 high-accident locations;
2. The district traffic personnel field-check all locations and submit remedial recommendations on 3 locations;
3. Headquarters reviews recommendations and approves them for implementation;
4. The district then implements them within 15 days; and
5. Headquarters performs a continuing review of the results and a benefit-cost analysis.

Missouri has been pleased with the results of this program. One major adjustment has been a change from 12 locations for each of the 10 districts to an unbalanced distribution to reflect the higher urban accident experience. The main objectives of the 120 Program are being fulfilled. Results of the program indicate an excellent benefit-cost ratio. A side benefit that is being realized is the acquisition of additional data regarding diagnostic and remedial information of the sign, signal, and striping techniques. The overall utilization and effectiveness of the Central Accident Data System have been accelerated by the demands made on it for the surveillance requirements of the program.

TECHNIQUES FOR EVALUATING THE EFFECTIVENESS OF GEOMETRIC IMPROVEMENTS MADE UNDER TOPICS

David A. Merchant, Federal Highway Administration

Program evaluation is conducted to determine whether a deficiency has been eliminated or lessened by a specific improvement. Projects for TOPICS usually are selected because of 1 or a combination of 3 deficiencies: capacity, safety, or operational traffic flow. These deficiencies are identified by analysis of volume, travel time studies, accident data, and vehicle-miles or vehicle-hours of travel for network situations. By a comparison of these indicators with "norms" for the particular location, problem areas can be identified.

The 3 categories of improvements include point, route, and network. For evaluation of point improvements, one may compute the new capacity under prevailing conditions and compare the new and old peak-hour volumes passing a particular point. Because delay is a good indicator of level of service, it may also be used for evaluating point improvements. Operational flow problems, such as poor signal timing, point obstructions, or need for a form of traffic control device, also may be detected from delay information.

There are a number of indicators for route improvements. Travel-time studies may point out the need for better signalization timing, more responsive equipment, or better progressions. The changes in the number of stops along a route and in total vehicle delay can show how well the improvement has succeeded. Accident data also may be applied to indicate needed safety improvements.

Evaluation of an area-wide or network improvement is probably the most difficult of all and is usually accomplished by expanding the results of the point and route improvements. This requires a great deal of consistency throughout the research activites. Some of the area-wide measures include the following:

1. Vehicle-hours in an area, based on the sum of vehicle-hours on all routes in the area;
2. Vehicle-miles of travel in the area;
3. Accidents or accident rates or both for the area; and
4. Cumulative number of stops and total delay time, or stopped time delay.

If before-and-after comparisons are made, these must be valid so that erroneous conclusions are not drawn.

An evaluation program should be developed for all TOPICS study areas. Even though it is not necessary to evaluate all projects, there should be sufficient evaluations to determine the effectiveness of TOPICS. In addition, complex and expensive projects should always be evaluated.

TRAFFIC CONFLICT CHARACTERISTICS AND ACCIDENT POTENTIAL AT INTERSECTIONS

Stuart R. Perkins, Traffic Improvement Association

Drivers, vehicles, and roadways are complicated co-contributors in traffic accidents. All three can vary significantly in character from one area to another, from one year to the next. To provide an understanding of the basic causes of accidents, recent research has been directed toward the determination of detectable measures of traffic characteristics that could conceivably develop into accidents. Numerous techniques have been tested, including continuous camera monitoring and near-miss accident criteria. These studies have shown that near-miss accident criteria are apparently highly subjective and nonrepeatable and, therefore, are unsuitable as traffic-measuring techniques.

Analysis of traffic accident reports at high-accident intersections has shown that the reported numbers of any particular type of accident circumstance are not large enough for adequate analysis. To objectively measure the accident potential of a given area, without the study of an accident history, the Traffic Conflicts Technique has been developed. When this method is used, an intersection can be evaluated completely in three 12-hour observation sessions, and the evaluation is more comprehensive than that resulting from a study of accident histories.

A traffic conflict is defined as any potential accident situation, including evasive actions of drivers and traffic violations. When confronted with an impending accident situation, a driver takes evasive action to avoid collision. Evasive actions of drivers are evidenced by vehicle braking or weaving, as attested by brake-light indication or lane change. Traffic violations, on the other hand, constitute a traffic conflict, or a potential accident situation, even when no other vehicle is in close proximity to the violation.

The Traffic Conflict Technique provides a relatively quick test for determining the effectiveness of traffic engineering changes by taking before-and-after conflict counts. Changes in road design, signing, signalization, environment, and accident trade-offs can be evaluated quickly and quantitatively. This technique has been used to define more than 10 specific categories of rear-end incidents. The Traffic Conflict Technique results in accurate measures of accident potentials, provides an understanding of the basic causes of accidents, and should ultimately lead to a reduction of traffic accidents. As a result of recent studies, the Traffic Conflicts Technique appears to be a very useful tool for traffic engineers.

DISCUSSION

Question

Is ambient traffic-stream speed at the study site taken into consideration in developing the correlation between accidents and conflicts?

Perkins

The actual approach speed of the traffic is not measured at the time of the conflict count. However, the speed limit for each approach to the intersection is recorded. Speed correlation is inherent in the data because drivers' judgment and reaction criteria are based on headway in seconds in which speed is a determinant.

Question

What is the relative importance of each of the 3 evaluation means or purposes covered by the panelists, i.e., cost-benefit ratios, research evaluation of improvement efforts, and demonstration of the effectiveness of these improvements to others such as the public and mayors of cities?

Perkins

Personally, I do not believe that one should assign relative importance factors to these evaluation techniques. They are each important in their own right. Cost-benefit ratios are necessary to assign priorities. Improvement evaluation methods, such as the Traffic Conflicts Technique, are necessary to point out specific problems that need correcting and to ensure that the problems have been solved after the improvement is made, without having to wait for a couple of years of accident experience. Demonstration of effectiveness of the improvement to the public and local officials is necessary to continue financial support for the projects.

Merchant

It would be extremely difficult to give the relative importance of the 3 purposes that have been covered because the overall reason is actually a mix of these 3 purposes. Which would be the most important would depend on the point of view of the person answering the question. As an engineer, I am very concerned about finding out just how well various kinds of improvements do work so that this information can be used by other people in designing an improvement most appropriate for a given situation. Some research people would think that research evaluation is most important. When budget time comes, there is no doubt that the ability to answer questions on the effectiveness of the improvements to those that hold the purse strings will be a very important factor. I do not believe it is possible to separate any one purpose as the most important.

Snider

All 3 evaluation means are closely related to one another to such an extent that the importance of one is difficult to weigh apart from the remaining two. Public acceptance of a project, we feel, is a strong evaluation point because in the public's mind the improvement has eased vehicle movement and should be directly related to a cost-benefit ratio. By this public acceptance, research evaluation can then be very wisely initiated to determine the technical aspects of this improvement for further consideration.

Mickes

While it is difficult to rate the relative importance of each of the 3 evaluation means or purposes, the most desirable yardstick will vary depending on the purpose of the program and the scope of the improvement. For the 120 Program, I would rate the 3 means as follows: research evaluation of improvement effects; demonstration of the effectiveness of these improvement measures to others such as the public and mayors of cities; and cost-benefit ratios.

Question

Why did you not cover in your presentation cost-benefit ratios as a federal requirement for TOPICS? If you eventually plan to come up with an economic analysis of TOPICS improvements, how will you ever be able to do it properly when needed data (such as time-delay data) are not collected before the improvement is made?

Merchant

Cost-benefit ratios were not specifically mentioned in my presentation because they are only one of a number of different methods to use in analyzing the economic effects of a TOPICS improvement. We have found from experience that, if we indicate a

method to be used, people tend to think that is the one method we want them to use and this is not true. In addition, we do not want to indicate that all evaluations would have to involve an economic analysis. We are hopeful that a number of them will; but in certain kinds of improvements, such as a pedestrian grade-separation structure for instance, we are not sure at this time of the best way to analyze the need. We do eventually plan to come up with an overall economic analysis of TOPICS improvements.

Question

Has any thought been given to a requirement for pre-evaluating proposed improvements before the money is spent and perhaps wasted?

Merchant

This is essentially a question as to whether the cost-effective approach will be formally applied for all projects. An engineer by training has to evaluate the situation before developing his alternatives, and he should select the best alternative before going into detailed design. Much of the time, this has been done rather quickly and without any formal documentation. In other more extensive and expensive improvements, the pre-evaluation should be quite extensive and probably should be formally developed. As an instance of this, I think any proposal for a computer-controlled signalization system should be backed up by some analysis to show why this kind of system is needed. However, because TOPICS improvements run the whole range in complexity and cost, I do not believe we should make any requirement for a formal pre-evaluation on proproposed improvements.

Question

To what end is all this evaluation information on TOPICS being collected inasmuch as TOPICS money has just been cut in half?

Merchant

Although the 1970 Federal-Aid Highway Act did drop the amount of TOPICS funds from \$200 million to \$100 million a year, there is still a backlog of funds available for FY 1970 and 1971 as well as a minimum of \$100 million a year for FY 1972 and 1973. I think all of us are convinced that traffic operational improvements will more than pay for themselves, but we have to be ready to prove that fact if we wish TOPICS or any other program to be continued.

Snider

Even though there has been a drastic cut in the amount of TOPICS funds, it is our belief that there are going to be future programs—such as the Urban Highway System Program that will be funded by the Federal Highway Administration—and, although these projects may be larger in scope, many of them will contain TOPICS type of improvements. Therefore, the evaluation information would be of significant benefit.

Question

I disagree with your contention that the type of data to be collected for evaluation of an improvement under TOPICS should be based on the purpose of the improvement project. Is it not true that you have to collect complete data on many things, beforehand, in order to tell whether negative results were produced by certain elements of the improvement, with an unintentional net decrease in safety or capacity?

Merchant

The overall purpose for evaluation is to find whether we got our money's worth. If the improvement was made to resolve a bottleneck, I do not want to add any unnecessary data in calculating the effectiveness of that improvement. If the improvement

was essentially on a route or network basis, we would be more concerned about some of the side effects, and the kinds of data developed for evaluation would reflect the broader effects. We are not, however, able to take all of these factors into account; in fact, we do not know all the factors that have to be considered. Therefore, even though it may require some over-simplification and may not permit consideration of all the possible side effects of a given improvement, we believe that overall evaluation should be based on whether the improvement did do what it was designed to do.

Question

Will we be given a chance to benefit from the negative results ensuing from evaluation of TOPICS projects? That is, will we be told what has not worked and why not? Are you going to admit what does not work?

Merchant

We hope to make available the data on effectiveness of these improvements, whether they are good or bad. We can find out a lot from the improvements that do not do what they were supposed to do, and we are willing to admit it if we find that something has not worked.

Snider

Our department has always worked on the premise that there will be a thorough and honest evaluation made of any type of project whether it be TOPICS, paint, or signal design. Therefore, if a negative result comes out of a TOPICS project, it would be reported and, to the best of our ability, so would the reasons why.

Question

Why did you seek to achieve level of service B or C in your TOPICS improvements and not level A?

Snider

In our TOPICS improvements, it was not intended to imply that we did not try to achieve level of service A in our design, but because of the economics this became impractical for we felt we had to stay within the financial capabilities of the city.

Question

Inasmuch as any city will definitely spend some of its own money on improvements, with or without TOPICS money, which types of projects do you earmark for TOPICS money?

Snider

We do not select project specifications on a "we're going to do them anyway" basis. Our projects that are selected have higher priorities when they are a part of a continuous route improvement. This is because we can improve the problem areas earlier, for theoretically we would have twice the amount of money because of these projects.

Question

You stated that benefit-cost ratios in your 120 Program were based solely on accident cost comparisons for the one year preceding and the one year following the improvements. Do you not consider benefits other than accident reductions?

Mickes

For the purpose of our 120 Program, only accident costs were included in the benefit-cost ratio. We did realize other benefits. However, because the 120 Program

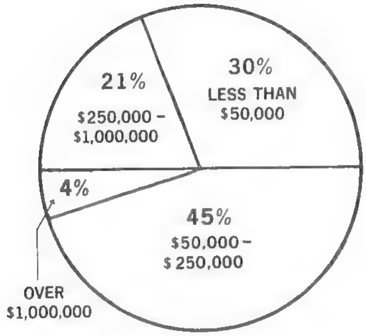

Figure 1. Distribution of TOPICS projects by percentage of total number of construction projects.

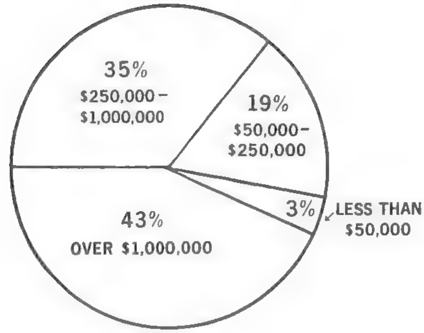

Figure 2. Distribution of TOPICS improvements by percentage of total cost.

is principally a low-cost safety improvement program, we did not include these other benefits.

Question

We used certain accident cost data in a study in Washington several years ago. Did you use the same sort of data in your 120 Program?

Mickes

We realize that there have been a number of studies conducted by various states regarding accident cost data. For the purpose of our study, we chose to utilize the National Safety Council's data.

ADDENDUM

In May 1971, the Federal Highway Administration reissued Policy and Procedure Memorandum 21-18 for TOPICS. This memorandum effected 2 major changes in the procedure for obtaining TOPICS funds. First, the requirement for cities to have a transportation planning process to be eligible for TOPICS money was eliminated for cities with populations under 50,000. This change reflected the intent of portions of the 1970 Federal-Aid Highway Act. Second, the May 1971 PPM gave the FHWA division engineer authority under certain circumstances to waive some requirements normally

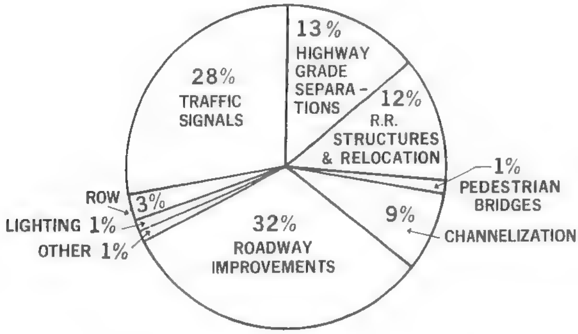

Figure 3. Distribution of TOPICS improvement costs by types of improvements.

applied to federal-aid projects, such as public hearings. He can also determine that environmental impact studies need not be done, and he can liberalize the project procedures requirements. These changes have cut approximately 1 year off project preparation and allow cities to proceed to construction quicker.

Initially lacking matching funds specifically earmarked for its use, the program has since grown to the extent that, today, more than 600 urban areas in all 50 states, Puerto Rico, and the District of Columbia participate in TOPICS. The 3-year apportionment funding for this program is $500 million divided among fiscal years as follows: 1970, $200 million; 1971, $200 million; and 1972, $100 million. As of September 1, 1971, $82 million has been authorized. Each state must obligate its share of the 1970 apportionment by June 30, 1972, or the funds lapse and cannot be used.

Items approved for TOPICS funds include signalization, channelization, pedestrian bridges, grade separations, railroad structures and relocation, lighting, off-street parking, and freeway surveillance and control. Figures 1, 2, and 3 show distributions of TOPICS projects and improvement costs.

It is obvious that more emphasis will be placed on this program in order to alleviate urban problems.

GUIDELINES FOR THE INCLUSION OF LEFT-TURN LANES AT RURAL HIGHWAY INTERSECTIONS

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Iowa State University of Science and Technology

The design of rural at-grade intersections is often referred to as an art rather than a science. The specific decision of whether to provide a left-turn lane is an example of the unavailability of a rational and objective approach to a major problem. This research has reviewed the various techniques and procedures in use, has measured traffic characteristics at typical Iowa intersections, and has developed a rational approach as a guideline for inclusion of a left-turn lane. The procedure is based on relating the road-user benefits to the cost of providing the added turning lane.

•THE AT-GRADE rural highway intersection is the weakest link in the process of planning and designing a highway. Increased vehicle operating costs, driver irritation, accidents, and all of the variously occurring operational inefficiencies are manifestations of the inability to maintain uninterrupted traffic flow conditions. According to the National Safety Council about one-fourth of all rural accidents occur at intersections (1).

In the typical design of rural highway intersections in Iowa, satisfactory highway capacity is generally not a limiting parameter. Through-traffic lanes easily accommodate all traffic desires. Thus, the analytical techniques applicable to high-volume urban areas have little application to this situation. Consequently, a very troublesome facet of intersection design is whether to provide an auxiliary lane for left-turning vehicles. This decision frequently dictates the extensiveness of the intersection development and is a determining factor in future operational aspects of the intersection. Thus, the designer should be as objective as possible.

A literature search was conducted in which prior efforts in this field were reviewed and analyzed. A few studies were found to be particularly germane; other studies formed a useful reservoir of background knowledge. Applicable studies were reviewed in detail (2, 3, 4, 5, 6, 7, 8).

To develop a rational approach to decision-making regarding inclusion of an auxiliary left-turn lane requires that certain fundamental questions be answered. A knowledge of traffic-flow characteristics at local intersections is necessary. The purpose of this study is to evaluate local conditions and develop guidelines for the highway designer (9).

PRELIMINARY EVALUATION

One of the first phases of this study was determination of desirable field measurement information. Thus, an early decision was required to ensure an adequate depth of data at the evaluation phase. Because of the limited time available, further post-analysis field study was not a probable possibility; consequently, the initial decisions were evaluated as completely as possible.

Physical site conditions as criteria for left-turn lane inclusion were the initial considerations. Included in this subject were items such as sight distance, roadway foreslopes, shoulder conditions and dimensions, alignment, grades, adjacent land use effects, and vehicle turning path inadequacies. In many cases adverse physical conditions may be a relevant factor as a measure of inadequacies of existing facilities.

Sponsored by Committee on Traffic Flow Theory and Characteristics.

Inadequacies represented by unsafe conditions should perhaps be a criterion by itself. In other words, there are 2 typical situations facing the highway planner: (a) an isolated intersection under consideration for a spot improvement program because of particular problems at the location, and (b) a highway improvement project that is of considerable length and that includes an intersection or intersections that must be evaluated regarding desired development. In the first case, the unsafe condition is very much a part of decision-making and is relative to establishing a project for improving inadequate physical elements. In the second case, existing substandard physical conditions are not relevant to the left-turn inclusion decision in view of the modern design standards that will automatically be utilized in the improvement project.

In either case it can be theorized that adverse physical site conditions will generate an improvement project. In most cases, however, the question of whether to include a separate left-turn lane is independent of the physical conditions. For example, there may be a special case where it is not feasible to develop desirable sight distance, and the situation may be alleviated by a left-turn storage lane. However, in the establishment of warrants for the inclusion of left-turn lanes at intersections, it was decided that substandard physical site conditions would not be included as a variable.

The factors to be considered and their relevancy were identified by asking the following questions: What are the adverse conditions at a rural intersection that can be expected to be alleviated with a separate turn lane? What factors measure these conditions? The answer to the first question includes delay to through vehicles stopped and waiting for a left-turner to select a gap and clear the through lane; delay to through vehicles decelerating from highway running speed and the subsequent acceleration to running speed; accident potential due to the left-turner decelerating, stopping, and standing in the through traffic lane; and reduction in the ability of the highway to accommodate the traffic demand within the service range desired.

Capacity is seldom of concern in the rural 2-lane highway situation under consideration. Consequently, it was determined that the investigation would be primarily concerned with vehicle delay and accidents as the 2 significant factors in establishing warrants for a left-turn lane. Measurement of these factors became the initial task.

INTERSECTION STUDY TECHNIQUE

A number of investigators have noted the problems associated with gathering and interpreting data regarding traffic performance at intersections (10, 11, 12, 13). The usual practice is to use 1 or 3 techniques, as follows:

1. A graphic recorder that has a moving paper on which as many as 20 pens denote the spatial arrangement of vehicles in relation to time by recording responses from input electrical signals provided by switches, pavement detectors, or signal controllers or by combinations of these;
2. Time-lapse photography in which all vehicular movements in the field of vision are recorded by a series of time-spaced 16-mm photos that can be later used to retrieve any particular characteristic of performance that can be visually identified; and
3. Observers with synchronized watches and stopwatches that record each vehicular event as it occurs during the study period.

In many cases a less detailed analysis of traffic performance may be required, and a simpler technique would be desirable. The authors observed the same problems that had been noted in the literature regarding the recording and retrieving of field data. A number of methods were tested in an attempt to hold the number of persons involved and the retrieval time to a minimum while maintaining reasonable tolerances. The technique finally adopted has not been previously employed in traffic operations measurement so far as the authors are able to determine.

In this method 2 unskilled observers using 2 inexpensive cassette tape recorders can obtain field data. One unskilled individual can reduce the data in a short time. Not only are equipment and labor costs reduced drastically, but also a more natural field study condition is maintained because of the unobtrusiveness of the observers.

The procedure is based on utilizing one tape recorder to play back a prerecorded signal of accurately spaced 1-sec clicks. This background time reference is played at

the site from the first recorder, while the second tape recorder records the traffic events that are translated into audible form by 2 observers. The result is a tape that, when played back in the laboratory, yields a time band (which is referenced to real time) with interspaced coded sounds identifying specific traffic events. This procedure provides what might be termed an "audiograph," similar to a visual graph obtained from a 20-pen recorder. After the occurrences of the traffic events are related to a time, it is easy to determine gap and lag characteristics, headways, and delay time.

ANALYSIS AND EVALUATION

Distribution of Vehicle Headways

One objective of field data gathering was to determine whether significant error would be introduced by modeling traffic flow using some theoretical distribution. From the results of research reported previously by others, the actual distribution of headways in a 1-way traffic stream on a 2-way road often does not correlate closely with a distribution based on an assumption of random arrival of vehicles. If passing opportunities are unrestricted, vehicle arrival will be nearly random in accordance with a Poisson distribution, and headways will be distributed in accordance with a negative exponential expression. This situation would occur on a 4-lane road carrying moderate volumes of traffic. However, if opportunities for passing are restricted, as must be the case on a 2-lane 2-way road, platoons of vehicles are formed for which the speed is established by the lead vehicle. Thus, the number of closely spaced vehicles is greater, and the entire distribution of headways is different from the distribution were the arrivals entirely random.

For this investigation, field data on vehicle headways were gathered for 94 1-way traffic streams representative of peak-hour conditions. One-way rates of flow during peak 15-min periods varied between 148 and 732 vehicles per hour. These data were tested for conformity with a negative exponential distribution and with several Pearson Type III distributions. There was not significant agreement between the actual and the theoretical. The most substantial lack of agreement between observed data and a theoretical distribution for the frequency of occurrence of headways was with headways ranging from 1 to 3 sec. These actually occurred far more frequently than would be indicated by the theoretical distributions tested—the natural result of the formation of platoons of closely spaced vehicles. This lack of agreement is shown in Figure 1. The observed frequency of occurrence of headways of 4 sec or shorter is compared with that calculated by assuming a Poisson arrival process. Results using an Erlang distribution (a special case of the Pearson Type III distribution) are also shown. The research principals consequently concluded that further work based on an assumption of random arrivals would not be valid.

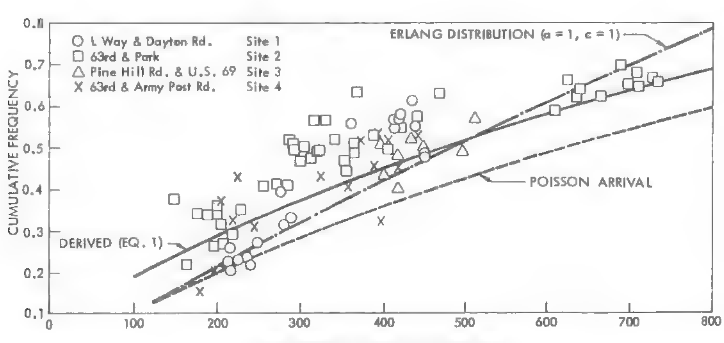

Figure 1. Frequency of occurrence of headways of 4 sec or shorter.

An alternative approach was to test the hypothesis that a satisfactory equation describing the frequency of occurrence of headways could be derived by multiple regression from the observed data. Such an equation, of course, would not be based on an assumption of randomness in the arrival process but would take into account the effect of platooning. It could then be used to calculate the probability of stops and the magnitude of delays caused by left-turning vehicles. The resulting equation is

$$y = 0.1279(t - 0.9)^{0.3681}q^{0.6094} \tag{1}$$

where

y = cumulative frequency of occurrence of headways equal to or shorter than t;
t = headway, sec; and
q = 1-way traffic volume, hundreds of vehicles per hour.

The coefficient of variation, R^2, for Eq. 1 is 0.79, indicating the appreciable amount of scatter that is common in samples of headway data.

An equation for the probability P of headway occurrence of any given length may be derived from Eq. 1.

$$p = (0.04708\, q^{0.6094})/[(t - 0.9)^{0.6319}] \tag{2}$$

Equation 2 is not unconventional in form in that P is a negative exponential function of the length of headway. While possessing some theoretical imperfections, Eqs. 1 and 2 satisfactorily reproduce the observed data for a wide range of traffic volumes and for the fairly narrow range of values of t that are pertinent for subsequent calculations.

A comparison of results using Eq. 1 with those calculated by assuming Poisson arrival illustrate the effect of platooning. This example is based on $t = 4$ sec and $q = 4.48$ or 448 vehicles per hour in 1 direction. Three samples from the observed data are also shown for comparison in the following:

Source	Value of y
Calculated from Eq. 1	0.484
Assuming Poisson arrival	0.392
Observed, site 1, 3-13-70	0.491
Observed, site 1, 4-28-70	0.473
Observed, site 2, 3-17-70	0.500

Lag and Gap Acceptance

A further objective for gathering field data was to determine lag and gap-acceptance characteristics for left-turning vehicles. With knowledge of these critical values and the probable vehicle headway distribution, we could estimate vehicle delay by appropriately considering the effect of queuing. Critical lags and gaps were determined from data gathered at each study site. These did not differ significantly from 1 site to another. However, sample sizes were quite small at sites 1, 2, and 4 so that values established for subsequent calculations were derived from a composite of the data gathered at all 4 study sites (Figs. 2 and 3). Results of this analysis are as follows:

Critical Lag or Gap	Sec
Lag	3.5
Gap for 1 car to complete left turn	5.5
Gap for 2 cars to complete left turn	7.3
Gap for 3 cars to complete left turn	9.5
Gap for 4 cars to complete left turn	11.6

Sample sizes for gap acceptance by more than 1 car were too small to be treated with a high degree of confidence. However, taken together they indicate rather clearly that vehicles are spaced at headways of about 2 sec when effecting left turns from a stop.

Theoretical Stops and Delays Versus Actual

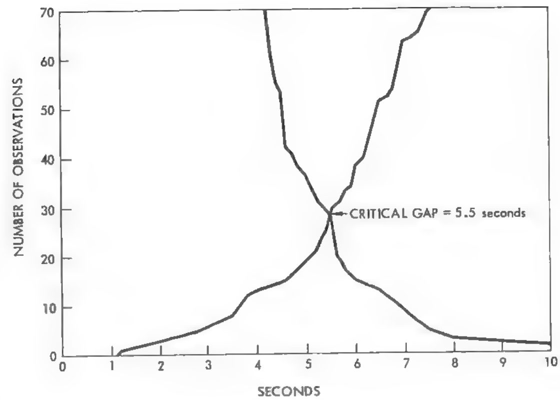

Figure 2. Consolidated totals for critical gaps.

With an expression for the spacing of vehicles in the traffic stream and knowledge of lag and gap-acceptance characteristics, one can calculate the probable number of vehicles that are forced to stop and the magnitude of delays to stopped vehicles. However, values calculated in this manner did not agree at all closely with those observed for the number of stops or vehicle delays. Significantly fewer vehicles stopped, and standing delay was markedly less than the theoretical values in nearly all samples. The research principals concluded that results from an approach based on this methodology could not be supported by the observed behavior of drivers at the test sites.

There are several possible reasons why human behavior might not conform with the expectations of a theoretical model in the situation studied. Lag acceptance, for example, is likely to be a function of several characteristics of traffic streams that are difficult or impossible to measure. A driver might risk acceptance of a very short lag if he observes a long line of traffic behind the car that will conflict with his left turn. On the other hand, he might reject a longer lag if the oncoming car is the only one visible to him; he knows that he will be delayed only a few seconds by waiting for it to pass. If sight distances are adequate, a driver approaching an intersection at which he is to turn left will adjust his speed in any of several possible ways according to his evaluation of vehicle spacing in the approaching traffic stream. He may speed up slightly and lengthen a lag to a level acceptable to him and complete his turn without stopping. Or, he may decrease speed slightly to avoid an unacceptable lag. He may avoid the necessity of stopping by completing his left turn immediately after an oncoming car has cleared the intersection. The angle and location at which a driver crosses the opposing lane also may be varied so as to reduce delay and the necessity of stopping. If the approaching side-road lane is not occupied, drivers will frequently initiate left turns early by turning at a flat angle and clearing the opposing lane before the arrival of an oncoming car. They may also delay their turns (without stopping) and then cross the opposing lane with a turn of very short radius to permit an oncoming vehicle to clear. Driver behavior, while difficult to predict with certainty, generally will be directed toward minimizing the amount of delay and the necessity for stops.

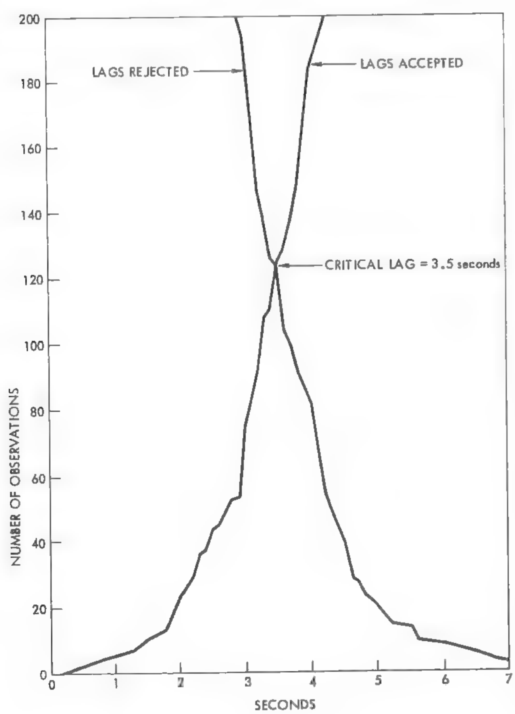

Figure 3. Consolidated totals for critical lags.

An Alternate Approach

Because the researchers were not able to accurately forecast driver behavior by using a theoretical model, the mass of observed data was examined again to determine its predictive ability. Multiple regression techniques were used to derive the following equations:

$$D = 0.04393\ q + 0.04901\ a + 2.147\ L \tag{3}$$

$$S = 0.007764\ q - 0.003546\ a + 0.3071\ L \tag{4}$$

where

- D = average standing delay for all advancing vehicles, sec;
- S = proportion of advancing vehicles that stop;
- q = 1-way volume of opposing traffic, hundreds of vehicles per hour;
- a = 1-way volume of advancing traffic, hundreds of vehicles per hour; and
- L = proportion of left turns in the advancing traffic stream.

R^2 values for these equations are 0.75 for D and 0.88 for S. The correlation matrix is interesting in that it indicates that D and S are much more strongly correlated with the proportion of left turns than they are with traffic volume in either traffic stream. The matrix is as follows:

Variable	q	a	L	D	S
9	1.00				
a	-0.27	1.00			
L	-0.65	0.51	1.00		
D	-0.08	0.41	0.59	1.00	
S	-0.26	0.36	0.71	0.83	1.00

The lack of significant correlation between the observed number of stopped vehicles and the opposing volume is surprising. This research was initiated with acceptance of an a priori assumption that there would be a direct and calculable relationship between stops and opposing volume. However, the observed number of stops differed markedly from values that could be expected in accordance with any theoretical distribution that the researchers could devise. Conclusions as to why this deviation could and did occur were reached only after a great deal of re-evaluation of the observed data and the techniques of analysis.

Conclusions as to why the number of stopped vehicles should be affected so strongly by the proportion of left-turning vehicles and so little by the opposing traffic are briefly summarized in the following.

1. Because of the occurrence of imperceptible or nearly imperceptible speed changes or adjustments in the location of the initiation of a turning maneuver, left-turning vehicles avoid a number of stops that are indicated as necessary by a theoretical relative positioning of opposing vehicles. Hence, use of theoretical spacings of opposing vehicles in combination with observed characteristics of lag and gap acceptance substantially overstates the necessity for stops and the magnitude of delays.
2. For purposes of this analysis, all stops were considered to be caused by left-turning vehicles forced to wait for opposing traffic to clear. Thus, it is logical to expect that the proportion of stops in the advancing traffic stream would bear a direct relationship to the proportion that turn left. This is true even though we may not be able to predict whether a given left-turning vehicle will be required to stop.
3. The total delay and the number of stops for all vehicles are calculated by multiplying Eqs. 3 and 4 by the advancing volume. Hence, these values are a function of a, and indirectly of q, where there is a reasonable directional balance in traffic flow. The advancing traffic stream constitutes from 30 to 70 percent of the 2-way traffic in the data from which Eqs. 3 and 4 were derived. We must assume that the effect of the op-

posing volume would be more significant if a greater imbalance existed and that Eqs. 3 and 4 would not then accurately predict stops and delays.

Research personnel concluded that Eqs. 3 and 4 would be more suitable than theoretical models for use in predicting stops and delays caused by left-turning vehicles at typical intersections in Iowa.

Equations 3 and 4 may be multiplied by the advancing volume to yield total hourly delay and total number of stops. However, q and a were assumed to be peak-hour volumes, so this calculation would be appropriate only for the peak hour. The average delay and proportion of stops will be somewhat less for all other daily periods. Appropriate factors for average hourly percentages of weekday traffic must be substituted for each of the 24 hours of the day in order to calculate daily totals. These factors were developed by the Iowa State Highway Commission from 54 automatic traffic recorder stations on rural primary highways in Iowa during the period from 1967 to 1969. The following equations in terms of daily traffic volumes result when these factors are used:

$$D_D = A_a(2.147\ L + 0.00002393\ A_q + 0.00002669\ A_a) \tag{5}$$

$$S_D = A_a(0.3071\ L + 0.000004228\ A_q - 0.000001931\ A_a) \tag{6}$$

where

D_D = daily standing delay for all advancing vehicles, sec;
S_D = number per day of advancing vehicles that stop;
A_a = 1-way volume of advancing traffic, vehicles per day;
A_q = 1-way volume of opposing traffic, vehicles per day; and
L = proportion of left turns in the advancing traffic stream.

The average delay per stopped vehicle is D_D/S_D.

Left-turning vehicles constitute a majority of those that stop and are delayed. However, some straight-through and right-turning vehicles are also delayed and required to stop behind vehicles waiting to execute a left turn. Construction of a left-turn lane will not change the number of left-turning vehicles that are required to stop and will have an insignificant effect on the amount of standing delay that they encounter. A left-turn lane will remove left-turn vehicles from the through lane so that straight-through and right-turning vehicles may proceed essentially without delay. Hence, a part of the benefit derived from construction of a left-turn lane is measured by a reduction in the number of stops and amount of delay accruing to through and right-turning vehicles as a result of left-turn maneuvers. Field data were analyzed to establish the proportion of stopped vehicles that proceeded straight through or turned right. This factor, K, is a quadratic function of L, as follows: $K = 0.6134\ L - 0.5744\ L^2$, $(0.0 < L \leq 0.8)$.

COST-BENEFIT COMPARISON

Reduction in Vehicle Operating Costs

Benefits to road users through reductions in operating cost and time were calculated for the following 2 typical situations that are representative of most rural intersections in Iowa:

Situation	Posted Speed Limit (mph)	Assumed Running Speed (mph)
1	70	55
2	55	45

The running speeds used are associated with a moderately congested level of service and are commonly used for analysis of operating conditions in Iowa.

Unit costs for passenger cars were assumed as follows:

Item	Amount
Value of time of vehicle occupants, \$/hour	1.85
Operating cost for idling during standing delay, \$/hour	0.11486
Operating cost for stop from 55 mph, \$/stop	0.03143
Operating cost for stop from 45 mph, \$/stop	0.01999
Excess time consumed per stop from 55 mph, hour	0.00584
Excess time consumed per stop from 45 mph, hour	0.00490

The value for time saved (14) and other values (15) are taken from other reports.

An equivalency factor is commonly used to account for the presence of trucks in the traffic stream. This is indicative of the average relationship of operating and time costs for commercial vehicles and for passenger cars. A factor of three to one is generally used by the Iowa State Highway Commission and has been used here. A quantity, T, is multiplied by the costs (or benefits) calculated for passenger cars to account for the increased costs (or benefits) occasioned by commercial vehicles. T varies depending on the percentage of trucks in the traffic stream.

Combining the costs for standing delay and stops, considering the effect of commercial vehicles, and converting to an annual basis result in the following equations:

$$b_{70} = K\ T\ A_a(5.160\ L + 0.00006991\ A_q - 0.00002443\ A_a) \qquad (7)$$

$$b_{55} = K\ T\ A_a(3.685\ L + 0.00004961\ A_q - 0.00001516\ A_a) \qquad (8)$$

where b_{70} and b_{55} are the annual reductions in operating and time costs for 70- and 55-mph posted speeds respectively. Pertinent costs are those associated with removing any necessity for stops by through or right-turning vehicles behind left-turning vehicles. The other variables were defined previously.

Most of the benefit calculated by Eqs. 7 and 8 is that occasioned by reducing the necessity for stops. The cost of standing delay (after stop) is typically less than 10 percent of the cost of stops from 55 mph and only about 13 percent of the running speed is 45 mph.

Accident Costs

A study was made of accident reports at the 4 rural intersections selected for field study. Records for the past 5 years were obtained but yielded a small number of accidents and a paucity of detailed information. The following information was desired:

1. The left-turn accidents that could be considered preventable (i.e., if a left-turn lane were available, the accident would not have occurred);
2. The estimated property damage per accident; and
3. The number and estimated cost of personal injuries.

Because of the extremely small sampling, no statistical significance can be associated with this information. However, generalized statements regarding the accident information are as follows:

1. About one preventable property-damage accident occurred per year per intersection;
2. About \$350 of reported property damage was estimated at each property-damage accident; and
3. About one personal-injury accident occurred every 5 years at each intersection.

In order to interpret the generalized accident information, based on the minimal Iowa accident rate conditions investigated, we reviewed further supplementary information. A number of studies have been conducted that assign a dollar value for the cost of various types of accidents. The National Safety Council (16) in 1965 established the following schedule of accident costs:

Accident Type	Cost
Fatal	$34,400
Nonfatal injury	1,800
Property damage	310

Included are wage loss, medical expense, overhead cost of insurance, property damage, and the indirect costs of anticipated future earnings for a death.

A study by Smith and Tamburri (17) reviewed prior research in Massachusetts in 1953, in Utah in 1955, and in Illinois in 1959. From research primarily based on the Illinois study, they upgraded the accident costs to 1968 California conditions and arrived at the following schedule:

Accident Type	Cost
Fatal	$9,700
Nonfatal injury	2,500
Property damage	500

Only the direct costs of a fatal accident are considered in their schedule, and this explains the difference with the NSC schedule.

Based on the local accident study and the accident cost assignments noted, the following accident cost schedule is established for this study:

Accident Type	Cost
Property damage	$ 500
Nonfatal injury	2,500

As a result of the accident investigations at the 4 study sites, it was determined that the preventable accident rate norm would be set at 1 property-damage and $\frac{1}{5}$ personal-injury accident per year. This decision yields $500 + $\frac{1}{5}$ $2,500 = $1,000 per year as a normal anticipated accident cost reduction. In the preparation of relative warrant equations and graphs, a provision is made for adjusting the norm results to reflect local conditions. Because of the rare occurrence of fatal accidents, the consideration of this type would severely distort the small samples taken in this study.

Accident cost estimation will normally take 1 of 2 forms: (a) an investigation of the accident records at the intersection under review, and (b) an estimation of the difference between accident rates that could be anticipated by a comparison to similar situation records and then forecast.

In the establishment of the relative warrants, the preventable accident costs at the field study sites were utilized. That is, $1,000 per year is the annual accident cost saving. However, provision has been made for using any value in the equation, or values of $500 or $1,500 in the graphical solution.

Highway Costs

It has been established that reduction of vehicular delays and accidents, by the construction of a separate left-turn lane, reflects a benefit. It follows that decision-makers will evaluate the cost-effectiveness of a left-turn lane design alternative in relation to the benefits that may be anticipated. The benefit-cost ratio has been selected as the method of evaluation for establishing relative warrants for left-turn lane inclusions.

A standard design for a channelized intersection has been adopted by the Iowa State Highway Commission in recent years. This design widens the normal 2-lane pavement width 16 ft to provide for a separate left-turn storage lane. Painted pavement markings are used to effect the channelization. The California left-turn lane warrant statement previously discussed (6) includes the following statements: "If the state highway is zoned for speeds of 55 mph or greater, the use of painted channelization should be considered. If the zoned speeds are less than 55 mph, the use of a physically protected form of channelization is suggested."

The estimate of construction cost used in this study is based on the difference between a normal 2-lane pavement in one case and a standard channelized intersection in the alternate case. The design is the standard Iowa State Highway Commission type shown in Figure 4. Unit prices reflecting current prices were obtained from commission contracts and right-of-way and maintenance departments. A summary of the construction and maintenance cost items is as follows:

Item	Additionally Incurred Costs of Channelized Intersection
Initial construction costs	
Pavement	$13,870
Earthwork	1,165
Drainage	336
Right-of-way	125
Lighting	9,000
Total	24,496
Annual maintenance costs	
Median and arrow painting	250
Rumble strip	250
Snow removal and salting	50
Lighting maintenance and energy	60
Total	$ 610

A very significant part of the capital costs is the initial investment is lighting. This design includes six 400-watt mercury-vapor luminaries.

The average annual project costs are estimated as the sum of the capital costs on an annual basis, plus the annual maintenance costs. The differential annual cost of a channelized intersection is calculated by the following equation:

$$\Delta C = (C_1K_1 + C_2K_2 + \ldots + C_nK_n) + \Delta M \tag{9}$$

where

ΔC = average annual cost difference incurred by the construction of a channelized intersection;
C_n = capital costs of individual construction items;
K_n = capital recovery factors for a specific interest rate and service life; and
ΔM = average additional annual maintenance costs incurred because of construction of a channelized intersection.

For the purposes of this study it was assumed that the service life and the interest rate were the same for each item of construction. The interest rate selected was 6 percent, which is currently used in the commission's planning division studies. The service life was selected as 20 years for every construction element. Consequently, the calculations of normal annual construction costs are ΔC = 24,496(0.087185) + $610 = $2,746.

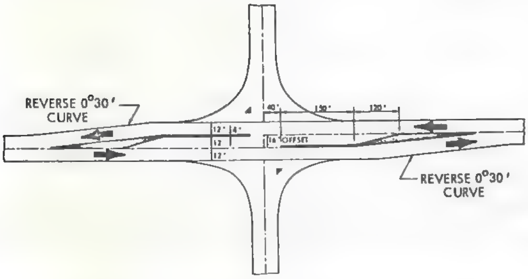

Figure 4. Typical primary road intersection.

Benefit-Cost Ratio

The benefit-cost ratio utilizes the savings from reduced stops and delays to through and right-turn vehicles (Eqs. 7 and 8) and from the elimination of preventable left-turn involvement accidents, C_a, as the benefit and the average annual project costs, AC, as the cost.

$$B/C_{70} = [K\ T\ A_a(5.160\ L + 0.00006991\ A_q - 0.00002443\ A_a) + C_a]/2{,}746 \quad (10)$$

$$B/C_{55} = [K\ T\ A_a(3.685\ L + 0.00004961\ A_q - 0.00001516\ A_a) + C_a]/2{,}746 \quad (11)$$

where B/C_{70} and B/C_{55} are the benefit-cost ratios for an area with 70- and 55-mph posted speeds respectively.

If the annual benefits exceed the annual costs (i. e., if B/C is greater than one), construction of a left-turn lane is warranted. Obviously other factors, such as safety or maintaining functional classification integrity, may in fact be dominant.

Equations 10 and 11 provide the highway engineer with a rational approach to decision-making regarding the added expenditure for a separate left-turn lane design.

SUMMARY AND CONCLUSIONS

An analysis of field data gathered under this project indicates that the use of theoretical distribution to describe vehicle headways is not applicable to rural 2-lane highways in Iowa. Distributions based on random arrivals do not correlate closely with actual 1-way traffic-stream data. An alternate approach was tested that uses multiple regression analysis of field data to describe the frequency of headways. Then, with a knowledge of lag and gap-acceptance characteristics, one can calculate the theoretical magnitude of stops and delays. However, values determined in this manner do not correlate at all well with observed data.

As an alternate approach, the mass of field data gathered were examined by using multiple regression techniques to yield equations for predicting stops and delays. Benefits accruing to road users by reducing stops and delays to through and right-turning vehicles were added to a potential reduction in accident costs. When they are compared to the added cost incurred by a left-turn lane construction project, a method of evaluating the cost effectiveness of the construction results.

The benefit-cost ratio technique is thus recommended as the criterion for decision-making. If the benefit-cost ratio is more than one, the construction is warranted. If less than one, the construction is not warranted (based on these factors alone).

ACKNOWLEDGMENT

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APPENDIX

WARRANTS FOR LEFT-TURN LANES AT 2-LANE RURAL INTERSECTIONS

For determining the benefit-cost ratio in a specific application, the following techniques are presented.

1. Benefit-cost ratio mathematical equations, Eqs. 10 and 11, may be solved.
2. Mathematical formulas, reduced to nomograph form (Figs. 5 and 6) may be used for repetitive applications. Three values for A_c are incorporated into the nomograph and represent a range on each side of the $1,000 norm value. The value of $2,746 for annual cost is incorporated in the nomograph, but any variation in this value may be used by multiplying the results by the ratio of $2,746 to the new value.

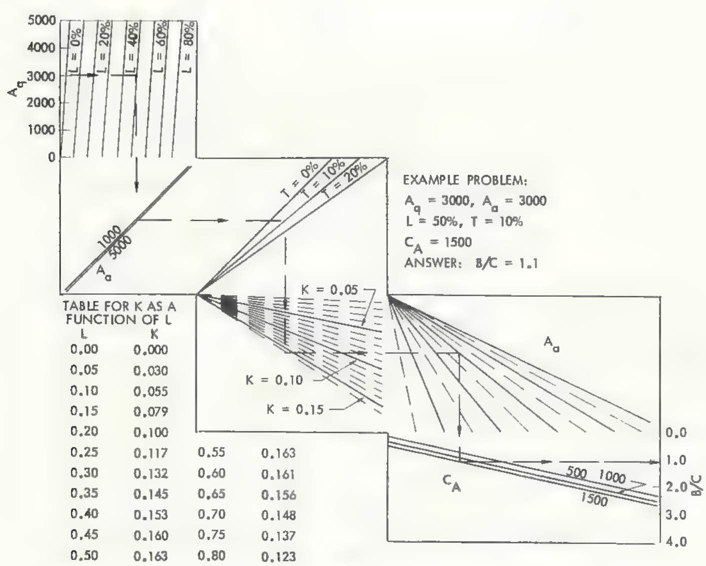
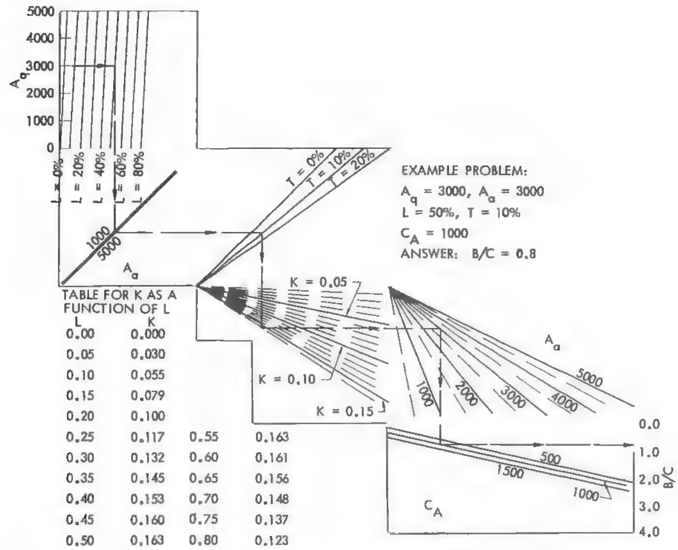


Figure 5. Nomograph for calculating benefit-cost ratio for left-turn lane—posted speed = 70 mph.



L	K	L	K
0.00	0.000		
0.05	0.030		
0.10	0.055		
0.15	0.079		
0.20	0.100		
0.25	0.117	0.55	0.163
0.30	0.132	0.60	0.161
0.35	0.145	0.65	0.156
0.40	0.153	0.70	0.148
0.45	0.160	0.75	0.137
0.50	0.163	0.80	0.123

Figure 6. Nomograph for calculating benefit-cost ratio for left-turn lane—posted speed = 55 mph.

3. A series of simplified charts (Figs. 7, 8, 9, 10, 11, and 12) for various posted speeds and accident cost savings, A_c, may be used for cases of equally distributed opposing and advancing traffic volumes, $A_q = A_a$. Shown on these charts are curves connecting the points where B/C = 1. Thus, the range above the appropriate truck percentage line warrants a left-turn lane whereas the range below does not warrant the construction.

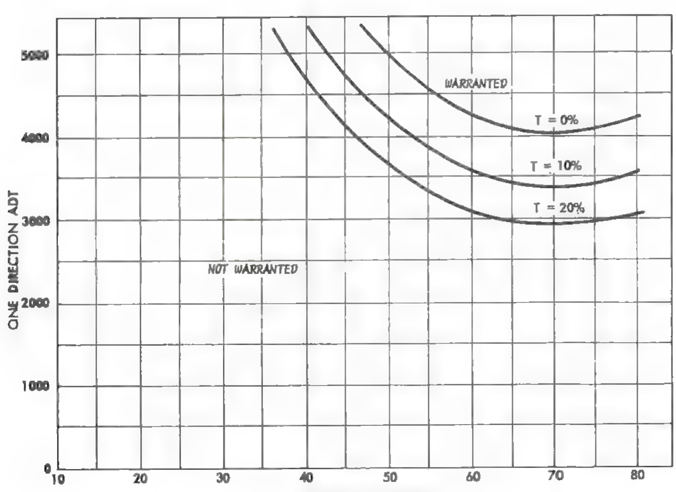


Figure 7. Warrant for left-turn lane—posted speed = 70 mph and annual accident cost reduction = $500.

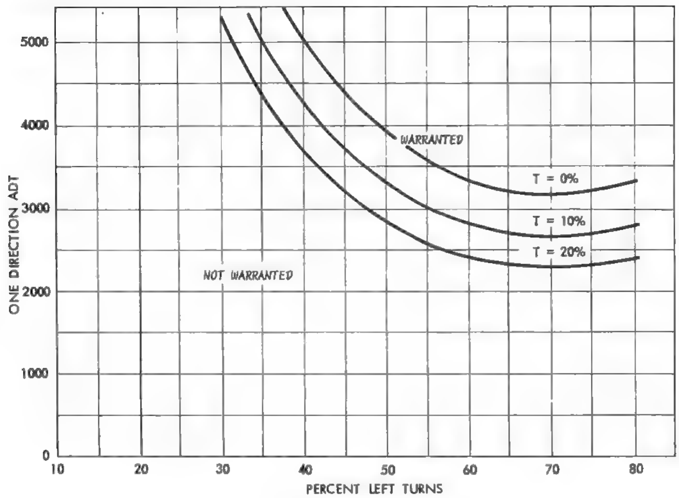


Figure 8. Warrant for left-turn lane—posted speed = 70 mph and annual accident cost reduction = $1,000.

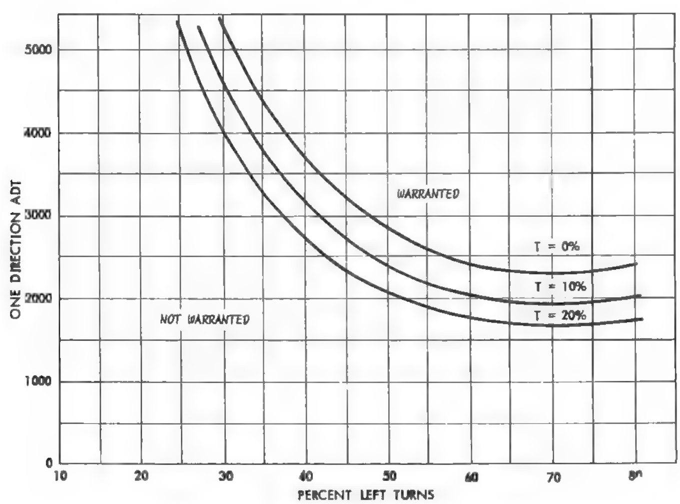


Figure 9. Warrant for left-turn lane—posted speed = 70 mph and annual accident cost reduction = $1,500.

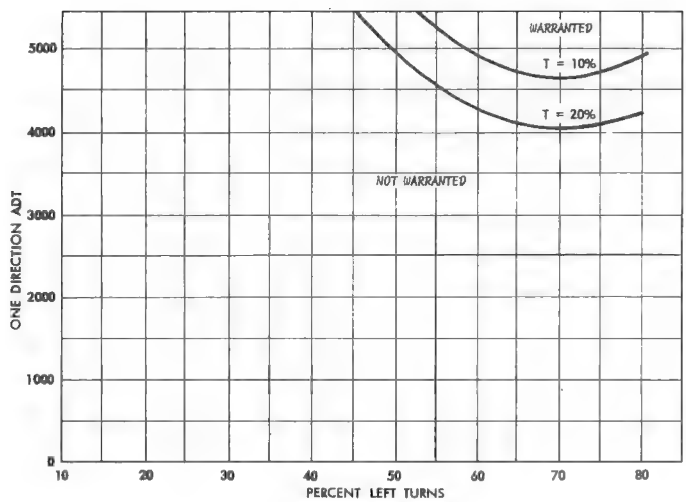


Figure 10. Warrant for left-turn lane—posted speed = 55 mph and annual accident cost reduction = $500.

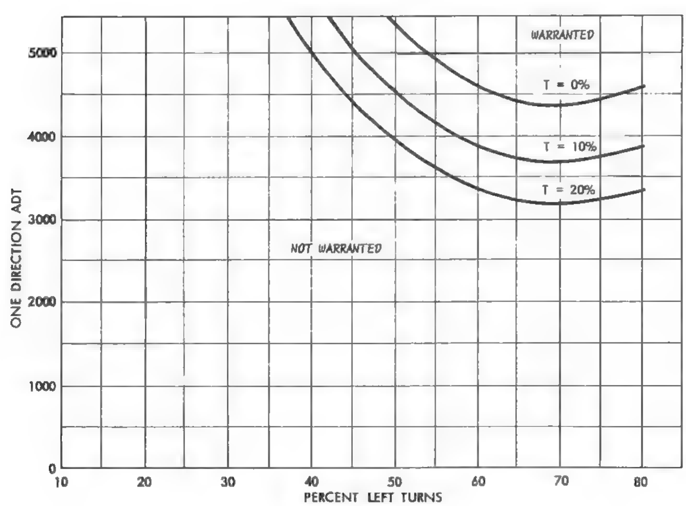


Figure 11. Warrant for left-turn lane—posted speed = 55 mph and annual accident cost reduction = $1,000.

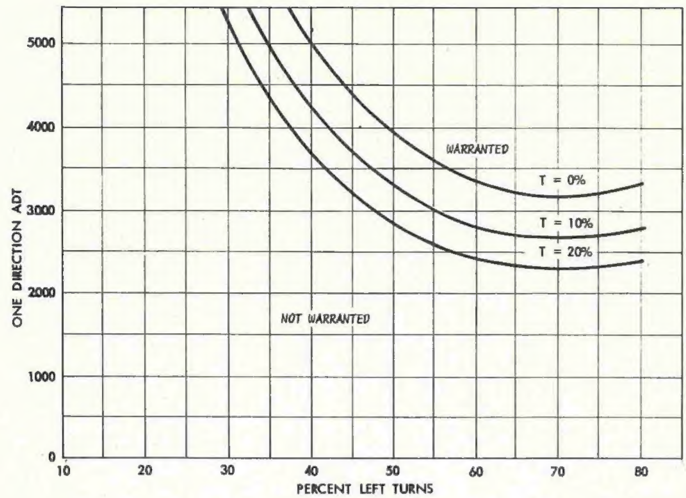


Figure 12. Warrant for left-turn lane—posted speed = 55 mph annual accident cost reduction = $1,500.

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